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PRESTRESSED CONCRETE INSTITUTE

Journal of the prestressed concrete institute

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Vol. 5, No. 2

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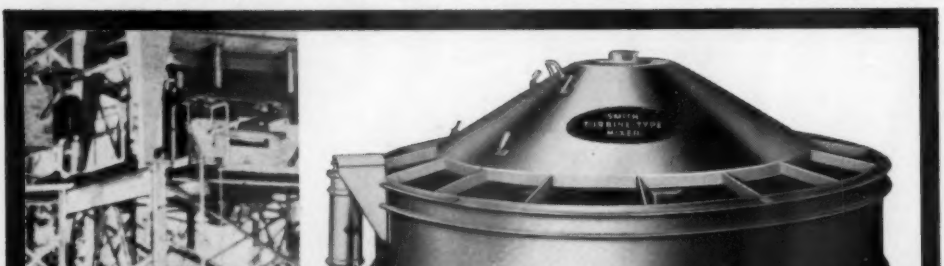

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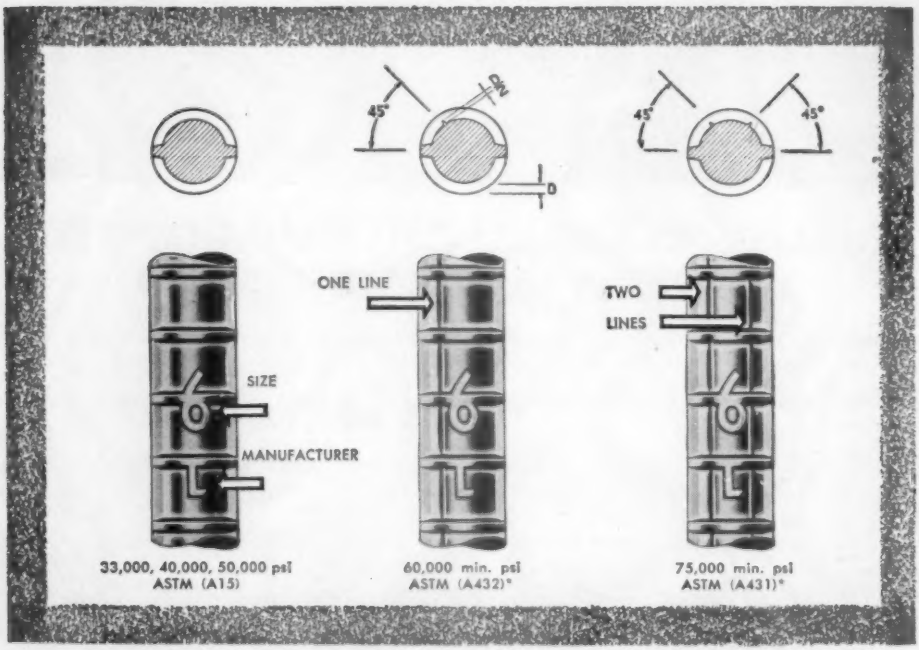

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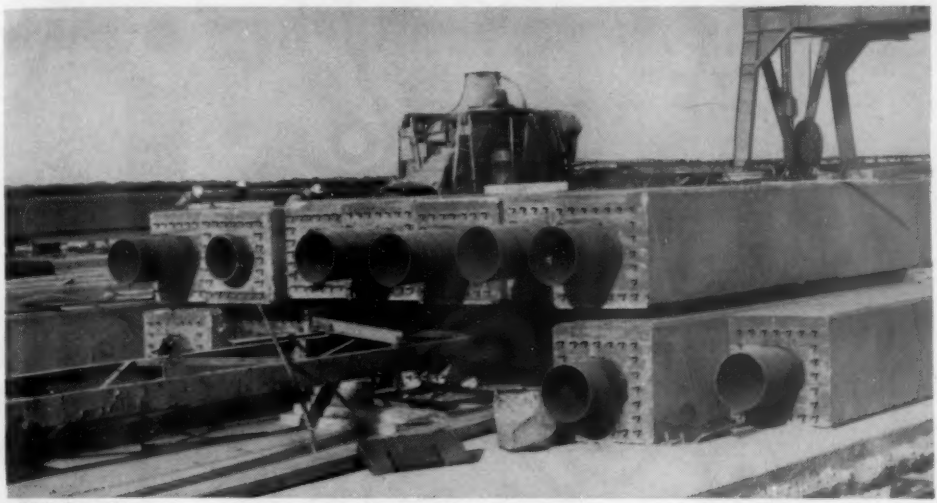

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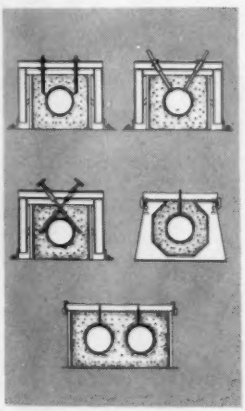

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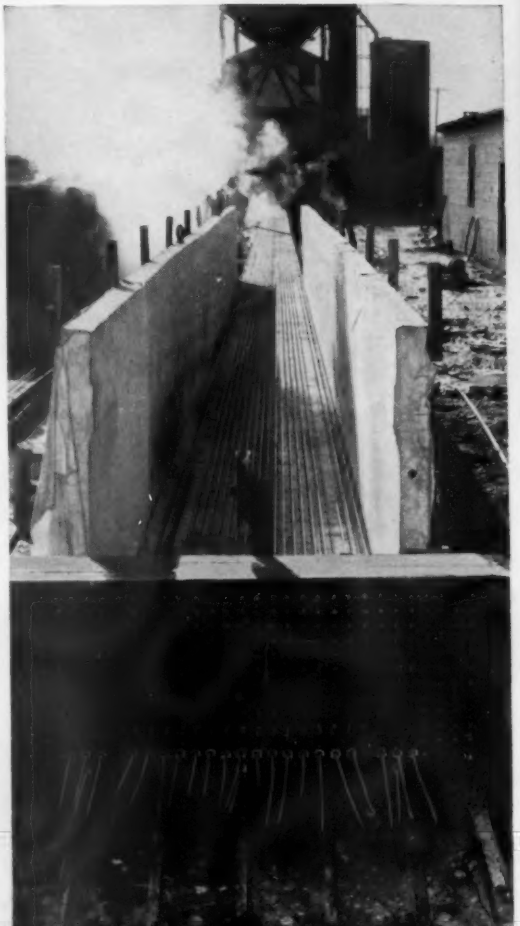

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PONTECORVO CANAL—Owner: Electric Company of Rome • Designing Engineer: Ing. Silvano Zorzi
Contractor: Farsura, both of Milan, Italy.

Flexible concrete plates drape-shaped to line canal bed

New technique uses prestressed POZZOLITH concrete

The Pontecorvo Canal carries water five miles to this hydroelectric project owned by the Electric Co. of Rome, Italy. Because the canal crosses irregular terrain, running partly in fill and partly in cut, a flexible concrete lining able to adjust itself was specified.

Inch-thick plates of prestressed concrete, four feet wide and up to 98 feet long, were used to line the paraboloid canal bed. A gantry crane gripped both ends of each precast plate, causing it to drape into a catenary curve when lifted. Placement and alignment of successive plates in the prepared mortar bed over the masonry base thus required a minimum of skilled labor. The joints were then sealed with non-shrink EMBECO® mortar.

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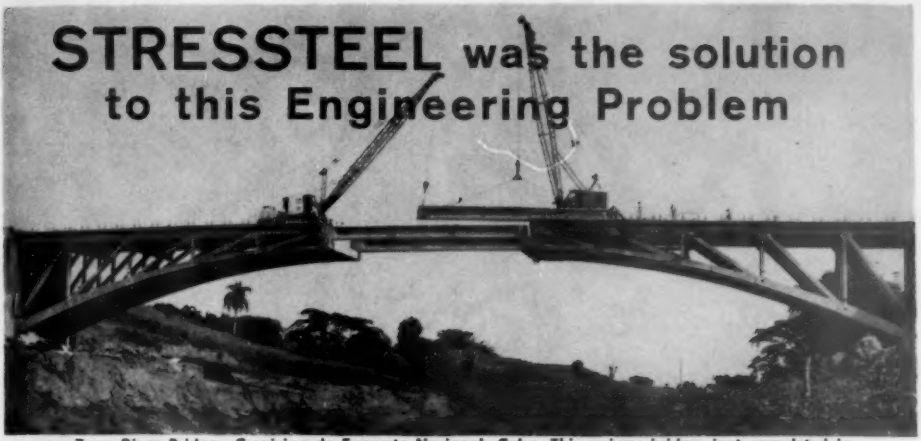

Zaza River Bridge—Comision de Fomento Nacional, Cuba. This unique bridge, just completed in Las Villas, for the first time uses precast and post-tensioned concrete to form cantilever trusses. Mario G. Suarez—Arango y Salas, Designers and Engineers. Cia. Territorial y Constructora Celo S. A., Contractor.

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Right—Assembling truss panels, with STRESSTEEL Bars forming the top chord of each truss

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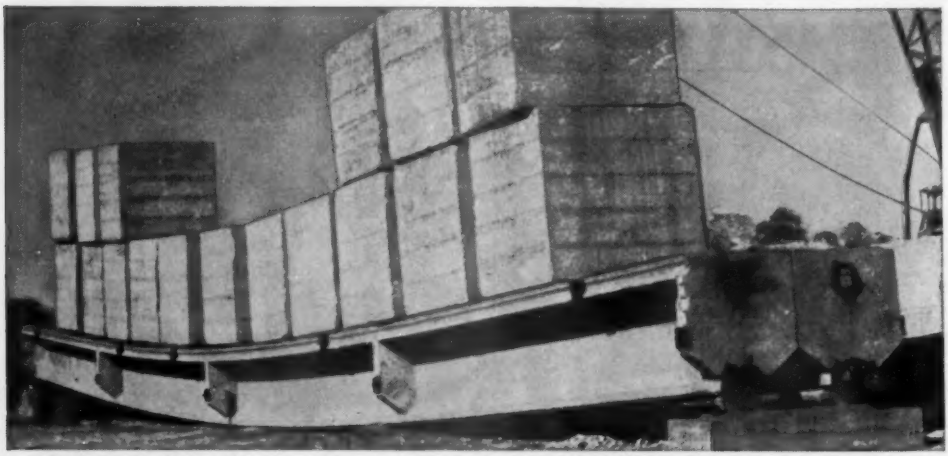

2 **153-TON LOAD**—more than four times design load. Deflection was 24 inches with no failure of concrete or strand. Test made for acceptability of 1400 girders for South Vietnam construction project.

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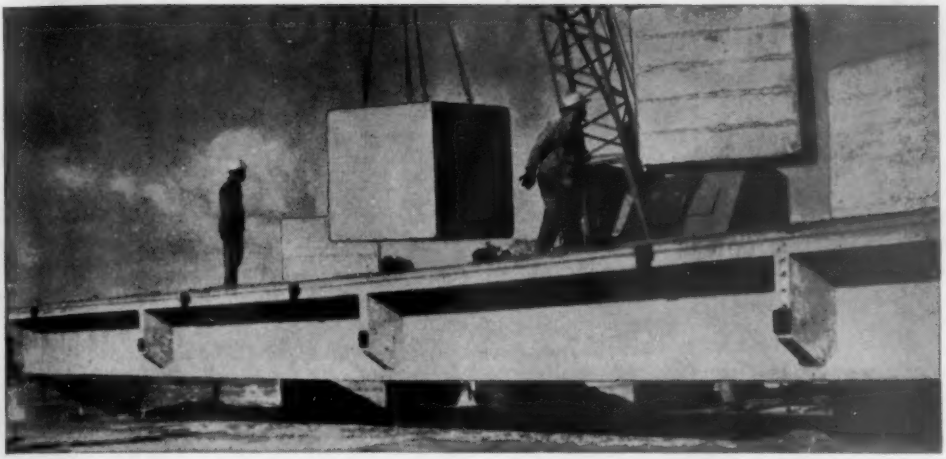
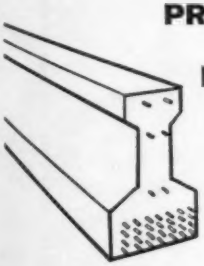
Prestressed concrete is unique. Its properties are a delight to creative architects and engineers. Thinner sections, longer spans, lower depth-to-span, longer cantilevering without ballast beams, reduction in weight, economy, architectural beauty. These and other advantages are a challenge to creative designers.



3 LOADS REMOVED—approximately 85% recovery of deflection immediately. Only hairline cracks evident. Designer—Capital Engineering Corp.; General Contractor—Johnson, Drake & Piper of Vietnam, Inc.; Technical Assistance—Freysinnet Co., Inc.

TALK WITH A NEARBY PRODUCER OF PRESTRESSED CONCRETE



He can discuss technical problems, point to examples of prestressed construction in your area, and advise on costs and delivery of beams from his yard.

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GUEST EDITORIAL

Architecture has been described as "the most practical of the fine arts and the finest of the practical arts". But architecture is also a science, and the measure of its success is often the measure of the degree of harmony achieved between science and art.

There is an important basic difference in the two which makes success in the practice of architecture extremely difficult, and indeed often makes it difficult to determine whether or not true success has really been achieved. For example, the basic science of mathematics presents problems which are capable of verifiable solution. Any number of students, given the same mathematical data and the same problem to solve, may all be expected to arrive at substantially the same answer. Architecture, on the other hand, cannot be reduced to formula; and solutions to problems which may prove quite satisfactory for one certain time, place and set of circumstances, may be a total loss for other times, other places, other circumstances.

For these reasons architects are continually searching, not necessarily for the correct answer to their problems, but for a better answer. We seek better materials, better techniques, better applications, better answers for the same problems we solved yesterday. Prestressed concrete, a most exciting element of this search, is certainly one answer for today.

It presents a new challenge to architecture for economy, form, and function. It is versatile, esthetic, and adaptable to creative design. Possessing all the advantages of reinforced concrete, it affords greater strength in less space, a commensurate reduction in weight, requires no maintenance. It may not be the only answer. But it is a good one. And it will be hard to beat.

JOURNAL in brief

A Challenge to the American Producer and Developer 14
W. E. Dean

The widely respected Florida bridge engineer tells producers that continued success will depend upon a high quality product at a low price. Engineers are also challenged to go beyond the low rise building and the short span bridge.

Load Distribution on a Prestressed Concrete Slab Bridge 18
A. C. Scordelis, W. Samarzich, and D. Pritz

A two-way prestressed flat slab bridge (60′ span, 100′ wide) is load tested with a 35 ton truck. Actual behavior compared rather well with the elastic theory but transverse prestressing had no noticeable effect on the distribution of moments.

Prestressing at Play 34
Walter L. Lowry, Jr.

The need for enlarging the Clemson stadium lead to the choice of prestressed concrete for the seating for economy and freedom from maintenance.

Restarch on Shear Strength of Prestressed Beams 40
M. A. Sozen

Research at the University of Illinois indicates that web steel is essential to allow prestressed beams to develop their full ultimate moment and deflection.

A Panorama of Prestressing in the United States 50
James D. Piper

The Walnut Lane bridge was built incorporating European techniques, but the American economy demanded that labor costs be reduced by mass production. Twenty-four photographs illustrate the U.S. development of prestressing.

Average Concrete Stress Along Prestressing Steel in Presrtessed Concrete Beams 61
Kenneth Kai-I Diao

In estimating prestress loss due to shrinkage and creep it is necessary to determine the average concrete stress along the centroid of the steel. This procedure is made easier by the use of formulas for typical cases.

Prestressed Concrete Airport Pavement 69
J. P. McIntyre

Prestressed concrete pavement has many important advantages over conventional pavement but to realize the full potential of this material more effective production methods must be developed.

Research on Fatigue and Shear in Prestressed Concrete 73
Robert F. Warner

Prestressed beams, like all other materials, will fail at a repeated load that is considerably lower than a static load. If cracks are prevented, it is reasonably certain that fatigue failure will not occur under working load.

Tentative Recommended Practices for Grouting Post-Tensioned Prestressed Concrete 78
Committee on Grouting of Post-tensioned Tendons

The industry has long needed a guide for grouting and these recommendations are presented for review and comment.

The Challenge to the American Producer and Developer

By W. E. Dean*

We have just heard descriptions of some of the unusual projects that are typical of Prestressed Concrete at Work in many areas of the world. They constitute a challenge to the American producer and engineer alike although, in listening to the work described by Mr. Piper, it seems that perhaps we have presented some challenges to the rest of the world. Certainly, prestressing in this country has advanced with seven league boots in the short interval of time since the World Conference in San Francisco in 1957. At that conference, one of the speakers closed his paper by likening prestressing to a youth just coming of age. In the two intervening years this industry has advanced so rapidly that we can flatly state that we have now attained a vigorous and lusty young maturity.

Two years ago one disturbing problem was the lack of engineers trained in the design of prestressed concrete. Experience gained in the past two years, the inclusion of prestressed design in the curriculum of most engineering colleges and numerous short courses have gone a long way toward the elimination of this problem. The development of our own AASHO-PCI standard bridge beams has done much to introduce simplicity and uniformity in the design of short span highway bridges and to shorten design time. This frees the engineer for the more complex aspects of design. I have no doubt that this, plus an ever growing store of both technical and construction experience, is providing better and better designs.

In these last two years, the Tentative Recommendations for Prestressed Concrete, the work of Joint Committee 323 of the American Concrete Institute and the American Society of Civil Engineers, has appeared and is being used as the basis of design for buildings and bridges. Although it is not a code and has, perhaps, a few shortcomings, it does provide a sound guide for design. A sub-committee of ACI is now putting it into code language and it is the basis of the tentative specifications for prestressed highway bridges that has been distributed by AASHO to all State Bridge Engineers. Railway engineers have provided for the design of prestressed railway bridges through the AREA Specifications.

From this new found maturity we can safely claim the acceptance of past challenges and much broadening of our future concepts. We are no longer thinking of 100-ft. spans as near the maximum in highway bridges. New York's Oneida Lake Bridge with its 320-ft. span proves this. We can design and build spans of this length and more. We have the ability and confidence so necessary for major structures. Two years ago it seemed that the multistory building market was best left

*Assistant State Highway Engineer (Structures), Florida State Road Department.

to reinforced concrete. But today we have ample use of prestressed concrete in two, three and four-story buildings. In Seattle, Washington the 21-story Norton Building is under construction and additional planning is certain to be underway for other tall buildings. But with this accomplishment of real maturity have come several challenging and disturbing elements. Some of these have always been with us, others are new problems.

To the Producer

Now that some of the romance has been rubbed off prestressed concrete by its intimate contact with the hard realities of day by day competitive living, we cannot ignore the fact that the quality of some commercially produced members often leaves much to be desired. This is not a blanket indictment of the industry but the frequency of complaints indicates that it is a serious problem that must be recognized. There are already cases of plants going out of business due to this complaint.

In prestressed concrete we have a competitive product. But the cost differential between it and competing materials is so narrow that the consumer just won't tolerate shoddy workmanship. It is usually poor workmanship that is cause for complaint, not poor engineering. Although not a manufacturer, I have seen many precasting plants, and the difference between good and bad workmanship is simply one of management attitude—not dollars and cents. If we're to maintain and develop this state of early maturity, steps must be taken immediately to insure a uniform high quality product throughout the United States. Publication of the PCI Standards for Prestressed Concrete Plants and the proposal for PCI certification of plants were steps in the right direction. They will gain more meaning when PCI plant certification becomes so recognized and accepted that it will be a requisite for the conduct of a prestressing business just as the Underwriters Laboratory approval is a requisite for the marketing of electrical appliances.

Some of your problems are those characteristic of any new industry. With inexperienced management, under capitalization will take its toll. But many plants which have not succeeded were those whose management ignored the fact that to exist today a business must offer an exceptionally good product or an exceptionally cheap one.

Now another important challenge. We all know that factory-produced prestressed products sell in a market that previously was almost exclusively a steel preserve. This is true in both the bridge and building products fields. This has been documented in various national publications such as a recent article in Engineering News-Record showing the results of a 13 State survey. This survey dramatically illustrated the increasing use of prestressed concrete against a decreasing use of steel.

Inroads into their markets have hurt the steel interests to the degree that several independent fabricators have actually entered the prestressing business. To be realistic, we must acknowledge that the steel industry in this country is dynamic and aggressive. It is not going to take this loss complacently. Right now their research people are working feverishly on ideas for new rolled sections that will be more competitive with prestressed concrete. Other ideas and plans for new

fabricated members at less cost are also being studied. In other words, the prestressing industry can expect to face a highly organized aggressive competitor. This competitor has both money and organization and it is facing you producers with your individually owned plants. And don't discount the importance of your competitor's collective programs of advertising and publicity. As yet the prestressed producers' efforts in these fields have been weak by comparison.

The prestressed industry is also facing other competition. The aluminum industry is aggressively entering the field of bridge and building parts and specialized structural applications. Reinforced concrete is proving more and more competitive. A shopping center in Minneapolis was recently reported to have a hyperbolic paraboloid roof with columns spaced at 47 feet for a contractor's cost of under $1 per square foot for shell and column. There's competition for your double T's.

Does this mean that we're going to see the decline of prestressed concrete? Of course not! What it does mean is that the producer must use every device in the industrial engineer's handbook plus his own ingenuity to cut production costs and raise the quality of his product. Frankly, I think you can do it.

To the Engineer

So much for the challenge to the producer; let's consider the engineer. Thus far, with a few notable exceptions, precast-prestressed concrete members have been designed and used as simply supported members. This is the principal reason you have been limited in a large degree to the low rise building and the relatively short span bridge. This has been a big market, one that will undoubtedly continue to take a large share of production. But for the use of prestressing to expand to a larger field, engineers need to devise some simple, inexpensive means to achieve continuity, also joints and connections that will permit frame action.

Continuity in bridges means a reduction in first costs and in maintenance costs by a better distribution of moments and the elimination of troublesome joints and extra bearings. Continuity and moment connections are the keys to the multistory building market. Research is underway at universities and such organizations as PCA on these subjects, but this is not enough. We need to channel all our talents into imaginative and practical solutions of these problems. You people interested in post-tensioning should throw your energies into the common effort. For unless we all work hard on these common problems it may be many years before proper solutions are found. This, then, is the main challenge facing the engineer. These are problems that should mesh with all facets of plant production and building erection. For the best ideas will be of no avail if they are not compatible with production and construction procedures. Mr. Collborg has shown examples of what is being done in Sweden; his connections appear to be very interesting and practical. Let's try some of them, or modify them to fit our particular conditions.

Two years ago at San Francisco it was stated that prestressed concrete was a youth, barely coming of age. Today it has reached an early and precocious maturity but it faces two major challenges, both rooted in a stiff competitive battle

with other structural materials. To the producer it means a hard, cold look at his production costs which will have to be kept to a minimum while maintaining or increasing quality. To the engineer it means much imaginative thinking to develop practical means for continuity and moment connections.

If this industry is to grow to full maturity and a secure place in the sun, the challenges must be accepted. I have the utmost confidence in this group and their ability to do so. This new industry, hardly nine years old, that has survived such a period of explosive growth as you have, has the youth, energy and drive to accept any challenge and fling it back to the source.

Load Distribution on a Prestressed Concrete Slab Bridge

A. C. Scordelis, W. Samarzich and D. Pirtz

This paper was outlined at the 1959 Prestressed Concrete Institute Convention, Miami Beach, Fla. by Prof. T. Y. Lin.

(1) Introduction

Analytical and experimental studies were made for the transverse distribution of wheel loads on a full scale simply supported prestressed concrete slab bridge. The slab is post-tensioned in two directions. Published results using the classical elastic theory were applied to obtain influence surfaces for longitudinal moments in the slab. Experimental values for longitudinal moments produced by a truck loading on the bridge were obtained, and these are compared with theoretical values. The effect of transverse prestressing on load distribution was also measured experimentally.

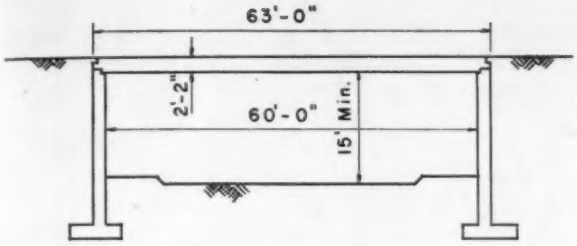
(2) Description of Bridge

The structure selected for the program was the Bacon Street Bridge in San Francisco, designed by the California Division of Highways. It is a solid slab bridge with a uniform depth of 2 ft.-2 in., a clear span of 60 ft.-0 in., and a total width of 99 ft.-6 in., Fig. 1.

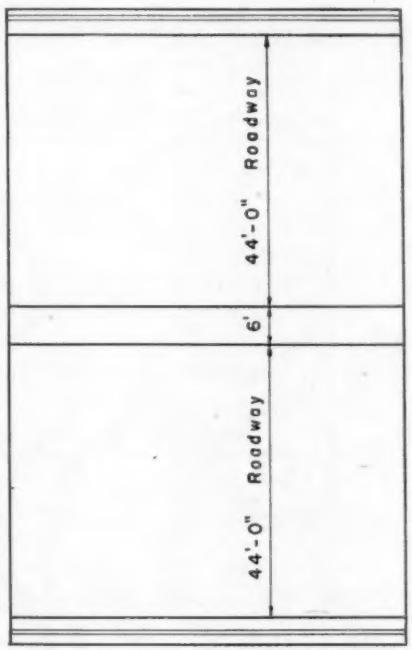
The bridge is prestressed longitudinally with 278 Freyssinet cables, each consisting of 18 wires of 0.196 in. diameter. These cables are draped parabolically, with the center of cables located at mid-depth over the supports and at 3 inches from the bottom of slab at midspan. Transversely, only 22 cables are provided, each consisting of 12 wires of 0.196 in. diameter. All transverse cables are placed at mid-depth and run straight throughout the slab. All cables are housed in flexible metallic conduits and were grouted after being tensioned. These cables produce an initial longitudinal prestress averaging 230,000 lb per linear foot of slab and an initial transverse prestress averaging 17,700 lb per linear foot. The effective prestress may be taken as 85 percent of the initial values, resulting in an average longitudinal compression of 675 psi in the concrete and a transverse compression of 48 psi.

The bridge is designed for six lanes of H20-S16-44 loading, with the load distribution based on AASHO Specifications for reinforced concrete slabs. AASHO Specifications for Highway Bridges and Bureau of Public Roads Criteria for Prestressed Concrete Bridges were followed in the design, except for minor modifications according to the practice of the California Division of Highways.

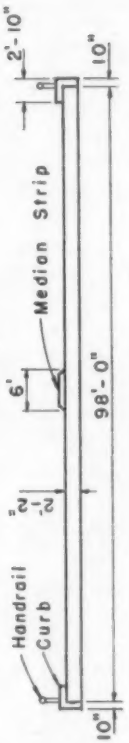
After the formwork had been completed, the cables and then the instrumentation were installed in the bridge. The slab concrete was



B. Longitudinal Section



A. Plan View



C. Transverse Section Through Slab

Fig. 1 - THE BACON STREET BRIDGE

placed on February 11, 1954. Longitudinal prestressing was begun on March 10 and completed on March 24. Transverse prestressing was begun on April 28 and completed the same day.

(3) Instrumentation

Twenty-four Carlson strain meters were placed in the concrete slab at six gage stations along the transverse centerline of the bridge, Fig. 2. Each station had 4 strain meters, with one pair located 1-1/2 in. from the top surface and another pair 1-1/2 in. from the bottom surface of the slab. Each pair of meters was placed so that one measured longitudinal strain and the other measured transverse strain. The

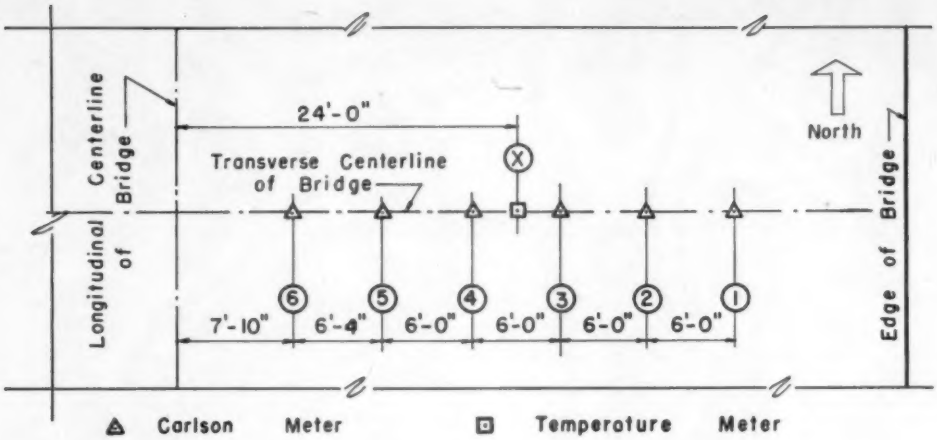


Fig. 2A - LOCATION OF GAGE STATIONS

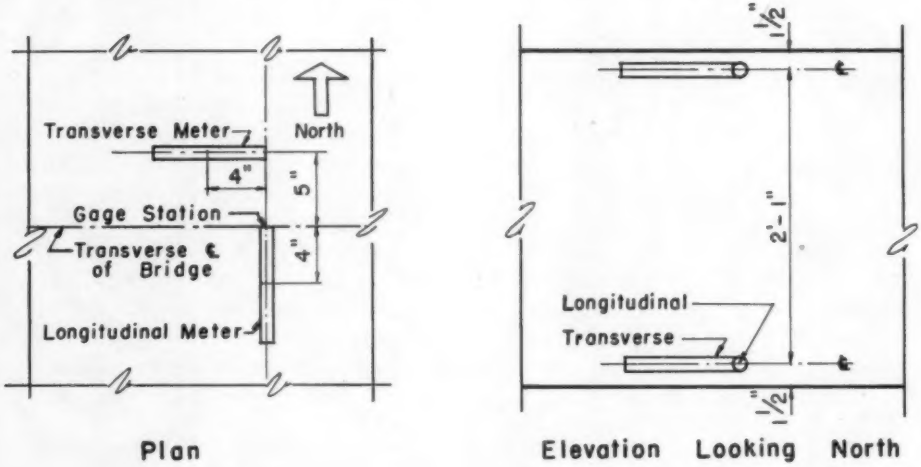


Fig. 2B - TYPICAL LOCATION OF CARLSON METER AT A GAGE STATION

leads from all strain meters were carried to a terminal box located in the curb of the bridge.

In order to obtain simultaneous readings on all gages for a truck in any position on the bridge, it was decided to use a procedure which was found to be successful on the San Leandro Creek Bridge Project.[1] This procedure consisted of taking dynamic recordings on the gages as a truck moved slowly across the bridge. This was accomplished by means of two oscillographs located in an instrument trailer stationed under the bridge. In addition several switching tapes were placed transversely across the bridge so that, as the truck moved across the bridge its position at any time could

be interpreted by the mark it left on the oscillograph record as it crossed one of the switching tapes.

For a truck moving at a speed of one to two miles per hour, the oscillograph records gave data corresponding to various static positionings of the truck. In this manner a great deal of data was obtained in a minimum amount of time. The only other measuring devices were five copper resistance type temperature gages located in the slab at an independent gage station. These gages were placed at various depths in the slab so as to obtain a record of the internal temperature variation.

(4) Control Tests

In order to determine the properties of the materials used in the bridge and to correlate strains with stresses, control tests were made on specimens of both the steel and the concrete. Specifications called for a 4000 psi concrete at 28 days. It was further specified that tensioning of the prestressing steel would not commence until the concrete had reached a compressive strength of at least 3000 psi.

The concrete mix used contained six sacks of Type I portland cement per cubic yard of concrete. Aggregate consisted of 38 percent sand and 62 percent coarse aggregate with a maximum size of 1-1/2 in. The water cement ratio was 5.64 gallons per sack. The slump averaged 3-1/2 in., as determined by Kelly-ball test.

Twenty-three 6 x 12-in. cylinders were tested at the University of California and the Division of Highways in order to determine ultimate strength, modulus of elasticity, and Poisson's ratio at various ages. These specimens were cured at the bridge site for six days and then standard cured in the laboratory until they were tested. Typical stress-strain curves are shown in Fig. 3. Ultimate strength increased with age as indicated in Fig. 4. Measured values of Poisson's ratio increased from 0.12 at 7 days to 0.16 at 40 days.

One sample of the 18-wire cable and one sample of 12-wire cable with anchorages attached were tested by the Division of Highways. The stress-strain relationship for each sample is indicated in Fig. 5. The 18-wire and 12-wire cables exhibited similar characteristics. The 12-wire cable had an ultimate strength of 263,000 psi and an elongation at rupture of 5.8 percent in 10 inches. The 18-wire cable had an ultimate strength of 262,000 psi and an elongation at rupture of 5.6 percent in 10 inches. Both cables indicated a fairly uniform modulus of elasticity up to a value of about

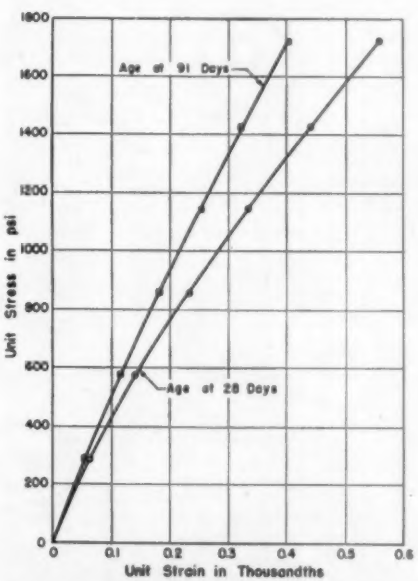


Fig. 3 - TYPICAL STRESS STRAIN DIAGRAM FOR CONCRETE CYLINDERS
(6 x 12 in. Cylinders, Cured at Site for 6 Days, Then Standard Cured in Laboratory Until Tested)

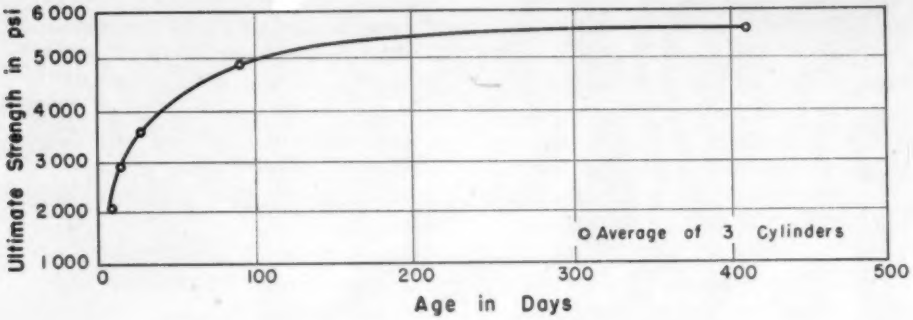


Fig. 4 - ULTIMATE STRENGTH OF 6 x 12 in CONCRETE CYLINDERS

158,000 psi. In the bridge they were stressed initially to 150,000 psi for the longitudinal cables and 140,000 psi for the transverse cables.

(5) Loading and Testing Procedure

The test vehicle selected was a 15 ton, Model 82FD, Rear-Dump Euclid truck. The truck was loaded with scrap concrete rubble. The total weight of the loaded vehicle was 69,400 pounds, with 56,800 pounds on the rear axle. An attempt was made to equalize the load on each of the front wheels and on each of the rear wheels, but because of the nature of the concrete rubble load, this could not be done exactly.

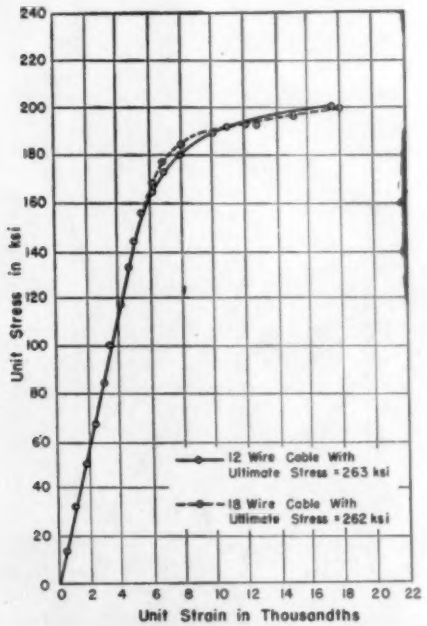


Fig. 5 - STRESS STRAIN DIAGRAM FOR 12 WIRE CABLE AND 18 WIRE CABLE

In order to determine the wheel loads standard highway loadometers were used. The wheel loads were weighed at numerous times during the test program, and the readings for a specific wheel load did not vary by more than 2 percent. As a check on the loadometer readings, the wheel loads were also determined by means of a public scale at the beginning of the test program. The appearance, dimensions, and magnitude of the loads for the vehicle selected are shown in Fig. 6.

The test program consisted of measuring the changes in strain in the various strain meters embedded across the bridge. The record of the change in strain was obtained by means of the oscillographs in the instrument trailer. One series of truck runs was made on April 26, with longitudinal prestressing having been completed but before any transverse prestressing had been provided in the bridge. This was called Series I. A second series of truck runs was made on April 29,

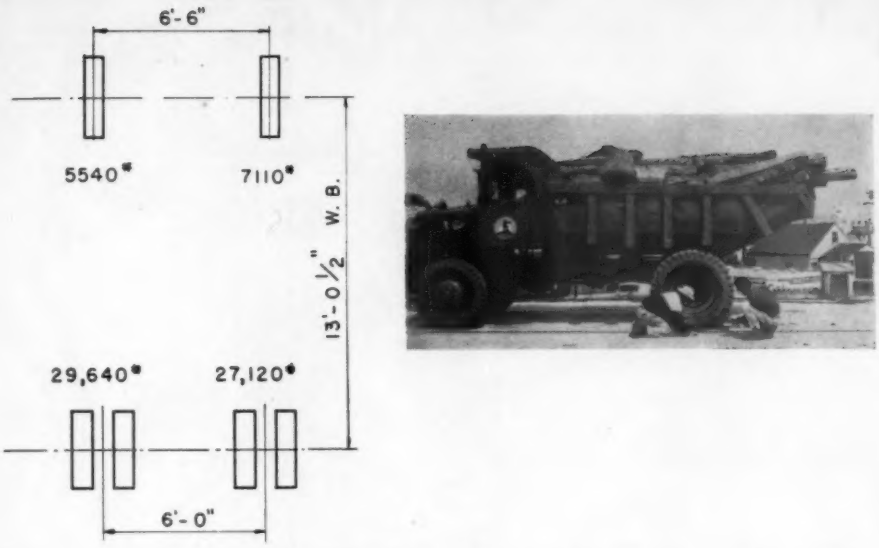


Fig. 6 - TEST VEHICLE LOADED FOR TESTING WITH WHEEL LOADS AND WHEEL LOCATIONS AS INDICATED

after all of the transverse prestressing had been completed. This was called Series II.

The paths followed by the truck as it proceeded across the bridge were exactly the same for the runs of Series I and II. In this way the effect of transverse prestressing on wheel load distribution could be determined by comparing the resulting data from the two series.

For each series ten separate paths were specified for the truck to follow. These were given letter notations of A through J. The location of these paths on the bridge surface are shown in Fig. 7. In each case the vehicle followed a longitudinal line marked on the bridge by a stretched rope so that the center of each of the rear wheels passed over the lines defining each path. It should be noted that paths A through E were so located that when the rear wheels of the vehicle were on the transverse centerline of the bridge, each rear wheel was centered almost directly over a gage station. This was done so as to obtain the maximum possible strain readings in the strain meters. At the conclusion of the test program, it was found that truck runs on paths F through J gave such small strain readings that the readings had little significance when the precision of the strain measuring devices was considered. For this reason the records obtained for paths F through J were not used for this paper. Runs A through E were each made first in a north direction, and then in a south direction.

All truck runs were to be made at an average speed of between 1 and 2 miles per hour. The speed of the truck was determined by checking the time it took for the vehicle to cross the bridge span. This time for all runs fell somewhere between 30 and 40 seconds which placed the speed of the vehicle between the

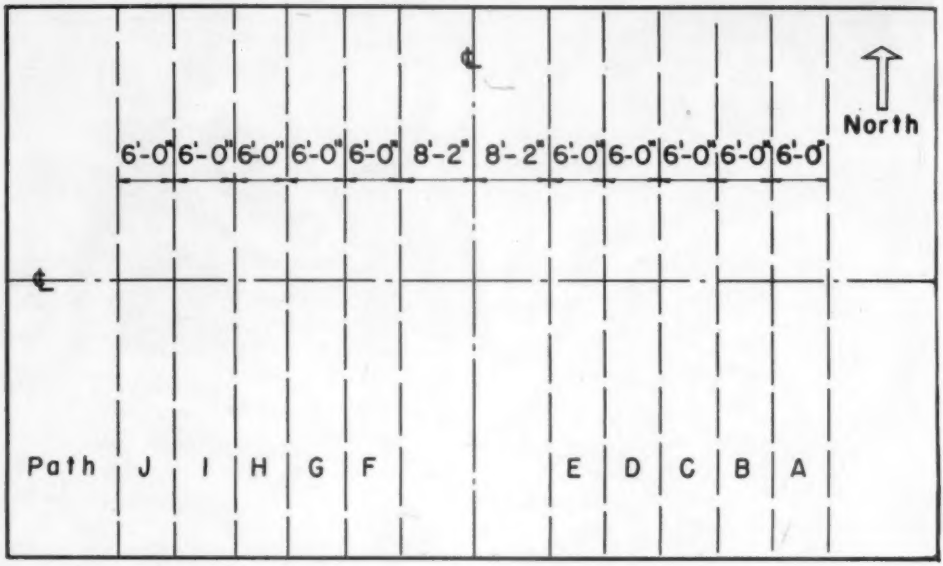


Fig. 7 - PATHS FOLLOWED BY TEST VEHICLE DURING TEST RUNS

desired limits.

(6) Theoretical Analyses

The problem of a slab, simply supported at its ends, subjected to concentrated loads has received the attention of numerous investigators.[2,3,4,5,6,7,8] An excellent summary of "Research on Highway Bridge Floors at the University of Illinois, 1936-1954"[9] also discusses related problems. This publication contains a set of references dealing with the overall problem of highway bridge floors.

The purpose of the theoretical analysis used in this investigation was to obtain the distribution of longitudinal bending moments across the transverse centerline of the bridge for various truck positions on the bridge. The following assumptions were made in the theoretical analysis.

1. The slab is a homogeneous, isotropic, and elastic structure.
2. The thickness of the slab is small in comparison with its lateral and longitudinal dimensions.
3. The middle surface of the slab is considered a plane surface before loading, and the energy stored in the slab after loading is due to flexural and torsional stresses only.
4. Any straight line drawn normal to the middle surface of the slab before loading remains straight and normal to the deflected middle surface after loading.

The above assumptions lead to what is commonly called the ordinary theory of flexure of slabs or the thin plate theory. This theory can be used satisfactorily in all regions of the slab except in the vicinity of a concentrated load for

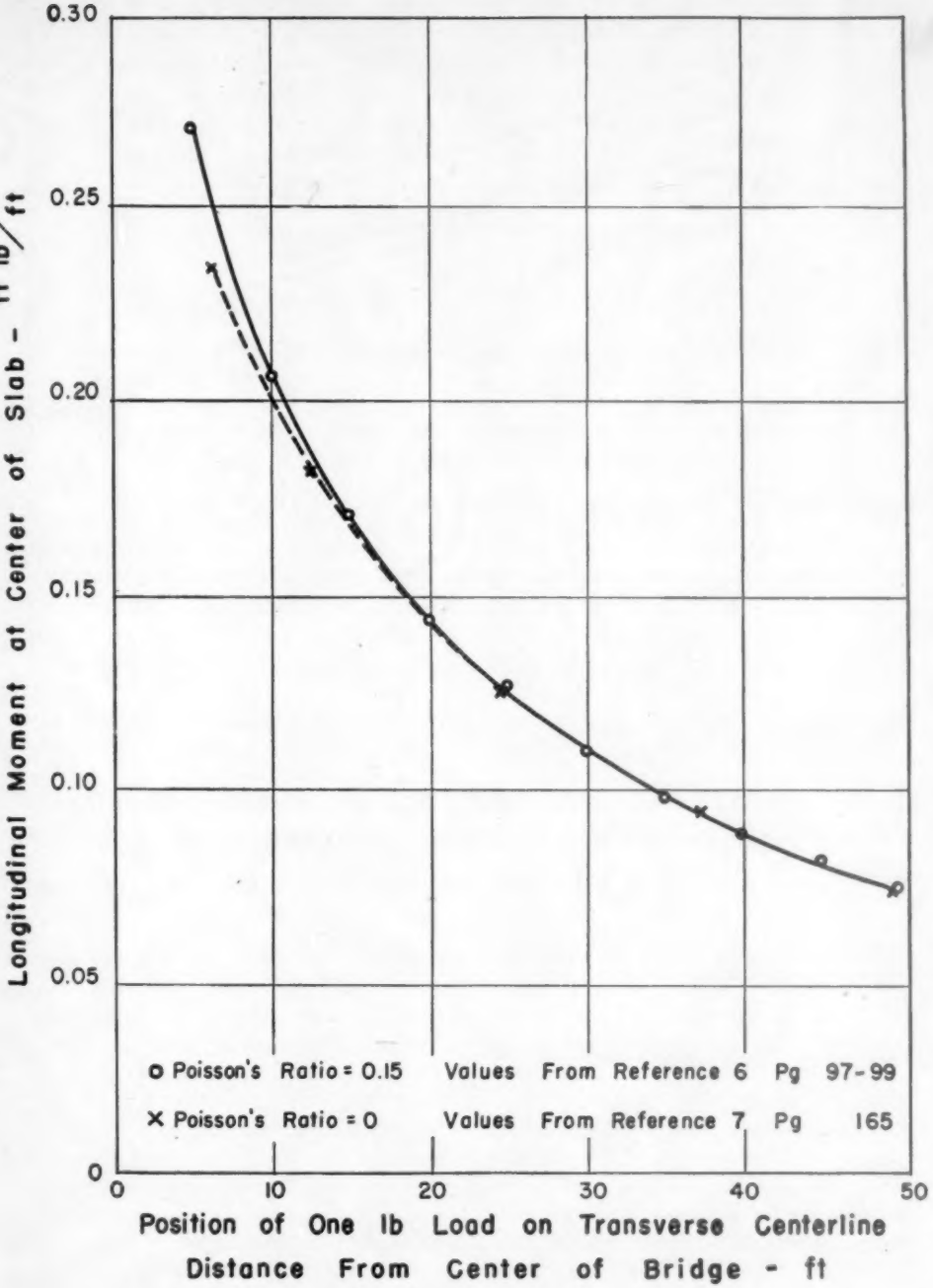


Fig. 8 - INFLUENCE LINES FOR LONGITUDINAL MOMENT AT CENTER OF BRIDGE

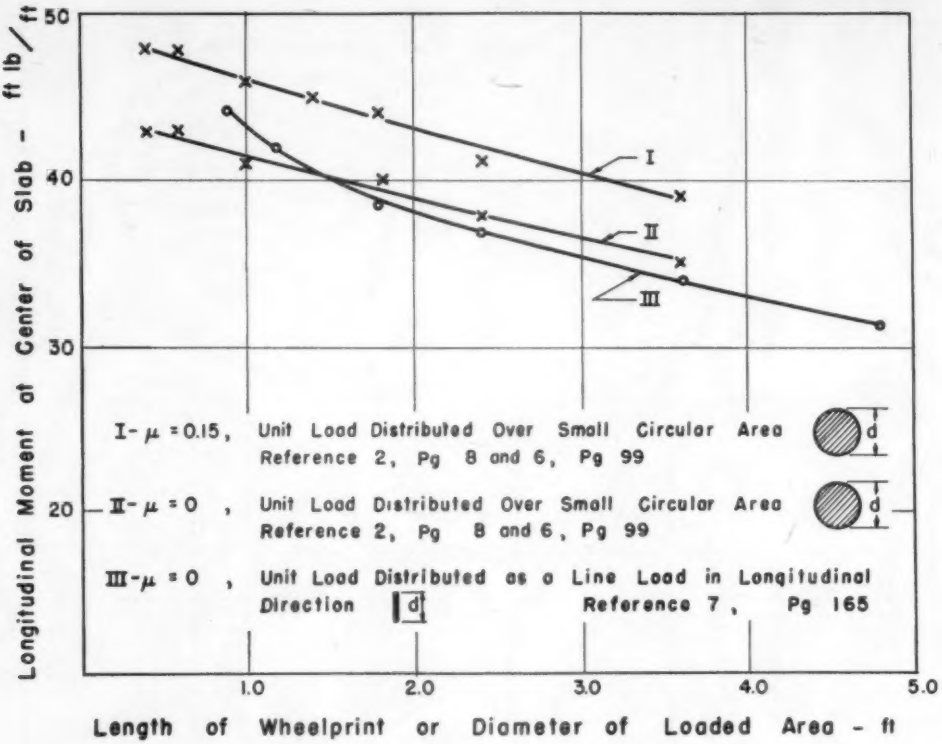

Longitudinal Moment at Center of Slab - ft lb / ft
50
40
30
20
I
II
III
I- μ = 0.15, Unit Load Distributed Over Small Circular Area
Reference 2, Pg 8 and 6, Pg 99
II- μ = 0 , Unit Load Distributed Over Small Circular Area
Reference 2, Pg 8 and 6, Pg 99
III- μ = 0 , Unit Load Distributed as a Line Load in Longitudinal
Direction d Reference 7, Pg 165
d
1.0
2.0
3.0
4.0
5.0
Length of Wheelprint or Diameter of Loaded Area - ft

Fig. 9 - EFFECT OF SIZE AND TYPE OF WHEELPRINT ON MOMENT DIRECTLY UNDER THE LOAD WHEN LOAD IS AT THE CENTER OF SLAB

which it indicates infinite moments and stresses. Westergaard[2] and Hall[3] overcame this obstacle by replacing concentrated loads by "equivalent uniform loads" on small circular areas which may be used in the ordinary theory of flexure to obtain correct results. Formulas are given for an "equivalent radius" in terms of the actual concentrated load. Olsen and Heinizhuber[7] replace concentrated loads by short line loads in the longitudinal direction and obtain solutions on this basis.

It is not the purpose of this report to present the mathematical analyses necessary for the solution of the general slab problem. For this purpose reference is made to the above-mentioned publications. The prime purpose here was to obtain theoretical results which could be compared with experimental results so that the validity of the use of the elastic theory for prestressed slabs of the type herein investigated could be ascertained.

The analysis was conducted in the following manner: A study of the available references indicated that influence surfaces for the longitudinal moments at the center of slab and at the edge of the slab

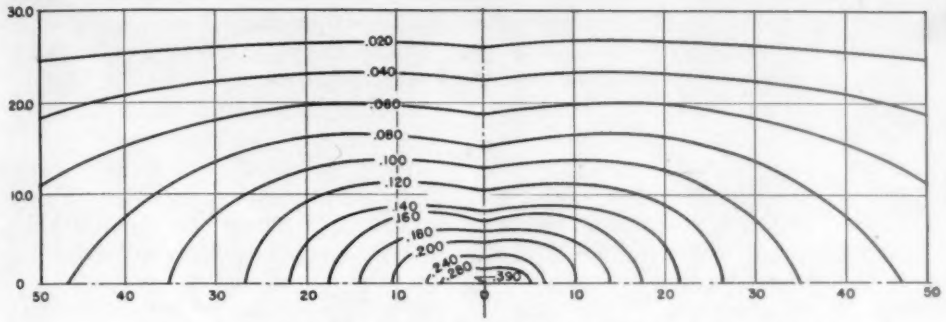


Fig. 10 - INFLUENCE SURFACE FOR MOMENT AT CENTER OF SLAB (Poisson's Ratio = 0 ; $L/b = 0.6/1$)

on the transverse center-line had been obtained by Olsen and Reinizhuber[7] for a slab having a span/width ratio of 0.6/1. which corresponded to the slab under consideration. These influence surfaces are shown in Figs. 10 and 11. Further investigations of various publications indicated that exact influence surfaces for longitudinal moments at points between the center and edge of the slab were not available and could only be obtained with considerable mathematical and computational effort. Since it was necessary to obtain moments at all points along the transverse centerline due to loads anywhere on the slab, it was decided that approximate influence surfaces for longitudinal moments at each of the experimental gage points on the transverse centerline would be obtained by interpolation between values available for the center and edge points. A study of the theoretical data available indicates that values thus obtained should be within 10 percent of the true theoretical values. It was felt that this would be satisfactory for comparison with experimental results.

Once these influence surfaces for longitudinal moments at various points along the transverse centerline were determined, the distribution of longitudinal moments along the transverse centerline for various

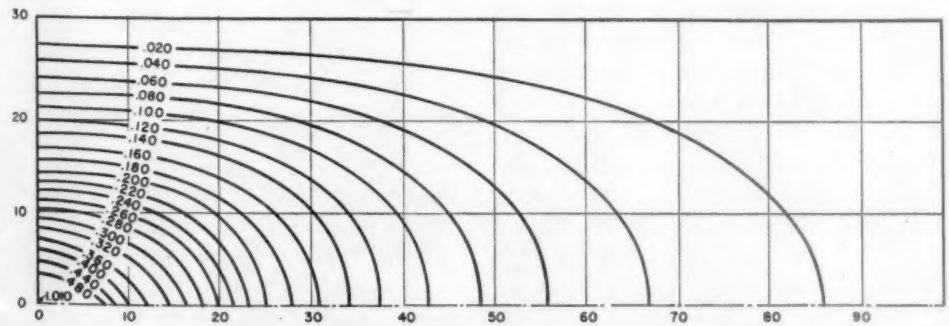


Fig. 11 - INFLUENCE SURFACE FOR LONGITUDINAL MOMENT AT EDGE OF TRANSVERSE CENTERLINE (Poisson's Ratio = 0 ; $L/b = 0.6/1$)

truck positions on the bridge was found by multiplying each wheel load by the proper influence ordinate and then summing the values for the four wheels to get the total effect. The resulting curves of theoretical moments are shown in Fig. 12-16. The line $x = 0$ represents the transverse centerline of the bridge.

Throughout the theoretical analyses, it was assumed that Poisson's ratio was zero. A comparison between the results for longitudinal moment at the center of the slab for Poisson's ratio equal to zero and 0.15 can be found in Fig. 8. It can be seen that there is little difference in the two curves for moment values.

When a wheel load is some distance from a point at which the moment is desired, the load may be assumed as concentrated or distributed over a small area, and the value of moment obtained will be the same in either case. However, when the wheel load is directly over the point in question, some assumption must be made as to the area of contact between the wheel and the slab. The effect of different assumptions regarding this contact area on longitudinal moment at the center of the slab may be seen in Fig. 9. It can be noticed that there is little difference between curves II and III. Throughout the analysis of the slab reported herein, it was assumed that the wheel print could be represented by a line load 1.8 ft. in length. This gives essentially the same effect as a load distributed over a circular area of 365 sq. in. It can be seen from Fig. 9 that even for relatively large differences from this assumed loaded area the moment is not appreciably affected percentage wise.

(7) Experimental Values

The experimental values of moments are obtained from the strain recordings of the Carlson meters. The strain values generally were between 5 and 40 micro-inches per inch, somewhat too small for high accuracy. Fortunately, the strain meters were quite sensitive and the smooth oscillograph recordings yielded much more reliable data than individually taken readings.

Strain readings due to live load taken before and after transverse prestressing were essentially the same. This indicates that the transverse prestress used in this bridge had no measurable effect on the distribution of moments due to live load.

The greatest difficulty in reducing the data lies in choosing the proper value of modulus of elasticity, used for converting the strains into stresses and bending moments. It was at first intended to use the tangent modulus of elasticity for concrete at the stress level produced by prestressing. Moments so computed yielded grossly different results from those computed by statics or the elastic theory. In order to resolve this problem, tests were conducted at the University of California to determine the modulus of elasticity of concrete which had been subjected to sustained high compressive stresses. These tests indicated that if the compressive stress in concrete had been maintained at a constant level for at least 28 days, additional compressive stress would yield a stress-strain relation the same as the initial portion of the unloaded concrete. Similar findings were recorded in a paper by Hrennikoff.[10] After considering the results of these auxiliary tests, it was decided that E_c

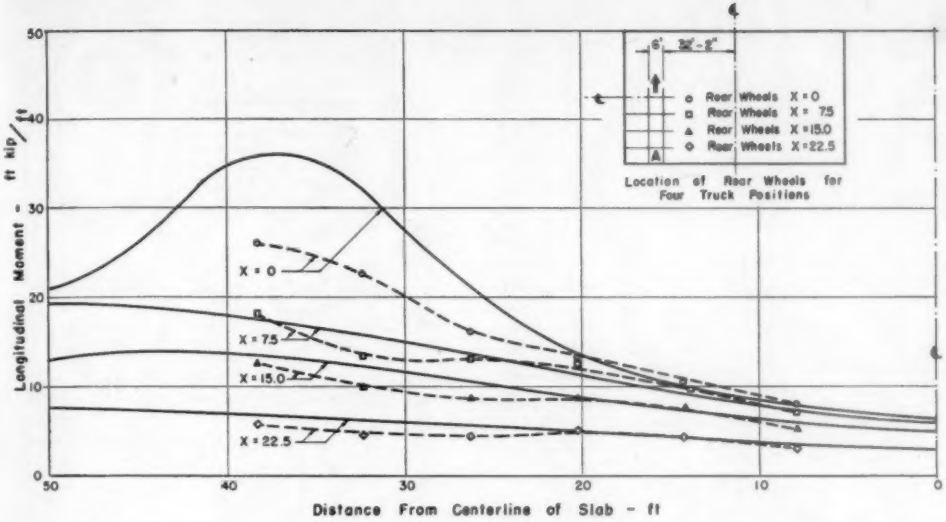

Longitudinal Moment - ft kip/ft
Distance From Centerline of Slab - ft
X = 0
X = 7.5
X = 15.0
X = 22.5
Rear Wheels X = 0
Rear Wheels X = 7.5
Rear Wheels X = 15.0
Rear Wheels X = 22.5
Location of Rear Wheels for Four Truck Positions

Fig. 12 - DISTRIBUTION OF LONGITUDINAL MOMENTS ACROSS TRANSVERSE CENTERLINE FOR FOUR TRUCK POSITIONS IN PATH A

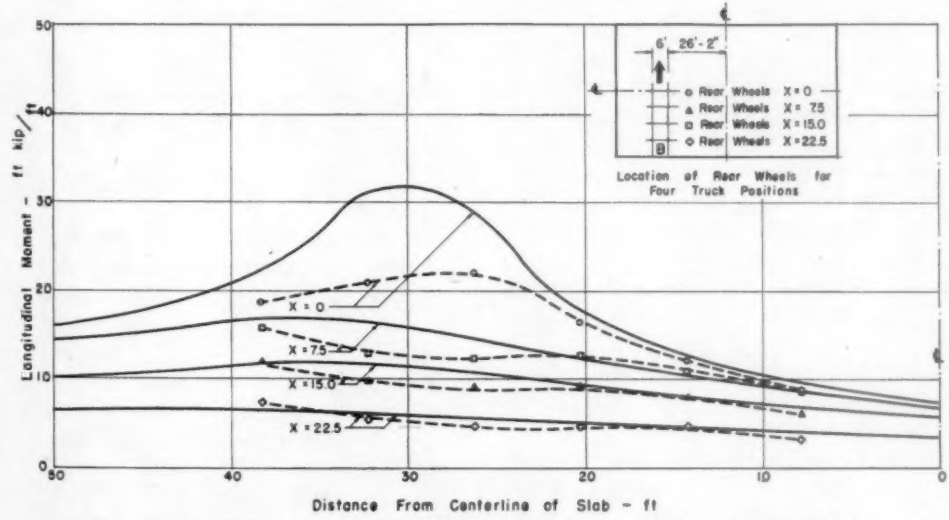

Longitudinal Moment - ft kip/ft
Distance From Centerline of Slab - ft
X = 0
X = 7.5
X = 15.0
X = 22.5
Rear Wheels X = 0
Rear Wheels X = 7.5
Rear Wheels X = 15.0
Rear Wheels X = 22.5
Location of Rear Wheels for Four Truck Positions

Fig 13 - DISTRIBUTION OF LONGITUDINAL MOMENTS ACROSS TRANSVERSE CENTERLINE FOR FOUR TRUCK POSITIONS IN PATH B

for computation should be taken as 5,000,000 psi for the strains produced by truck loadings.

Another discrepancy in strain readings occurred in the difference between the top and bottom gages. Computed from the neutral axis of the slab including the effect of prestressing steel, the top strains should be only 5 percent higher than the bottom strains. However, the experimental values recorded showed that the top strains were consistently 15 to 25 percent higher. This

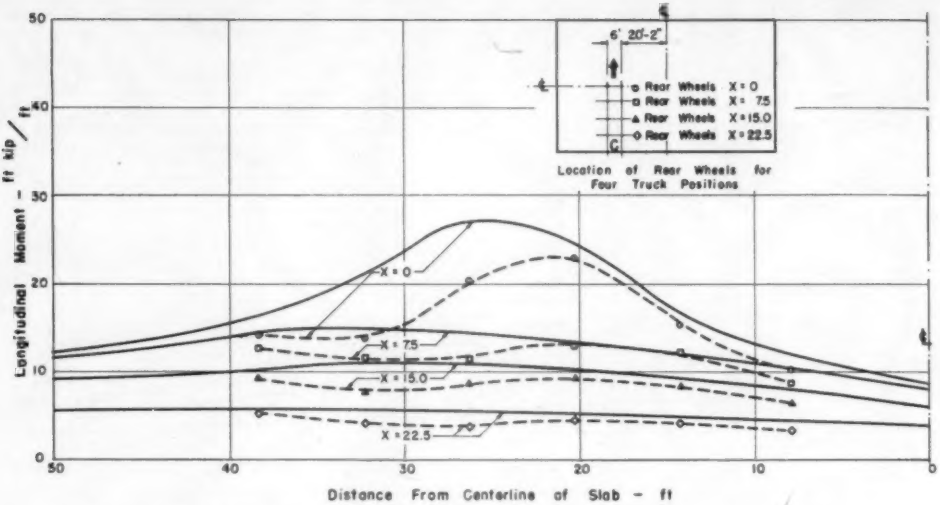

Longitudinal Moment - ft kip / ft
Distance From Centerline of Slab - ft
Location of Rear Wheels for Four Truck Positions
Rear Wheels X = 0
Rear Wheels X = 7.5
Rear Wheels X = 15.0
Rear Wheels X = 22.5
X = 0
X = 7.5
X = 15.0
X = 22.5

Fig. 14 - DISTRIBUTION OF LONGITUDINAL MOMENTS ACROSS TRANSVERSE CENTERLINE FOR FOUR TRUCK POSITIONS IN PATH C

cannot be satisfactorily explained, except that the top portion of concrete could be weaker than the bottom portion and thus possesses a lower modulus of elasticity.

The following procedure was followed in converting the strain readings into stresses:

1. Average the top and bottom longitudinal strains for both north and south runs.
2. Average top and bottom transverse strains for only the north runs.
3. Determine the longitudinal bottom stress σ_L in psi at strain gage level using the equation

$$\sigma_L = \frac{E}{1 - \mu^2}(\epsilon_L + \mu\epsilon_T)$$

where E = modulus of elasticity = 5,000,000 psi
ϵ_L = average longitudinal strains
μ = Poisson's ratio
ϵ_T = average transverse strains.

Then the moment in kip-ft per ft width of slab is computed from the equation:

$$M = \frac{\sigma_L I}{y}$$

where I = moment of inertia of section of a 12" wide section = 18,350 in.^{4}
y = distance from neutral axis to the bottom strain gage = 11.26 inches

$$M = \frac{\sigma_L (18{,}350)}{11.26\ (12{,}000)} = 0.136\sigma_L \text{ Kip-ft./ft.}$$

Typical results for data reduction using the above procedure are shown in Table I for the moments in the slab, which are plotted in Fig. 12.

Comparison of the experimental with the theoretical values is shown in Figs. 12 thru 16. Solid lines indicate theoretical values and dashed lines experimental values.

(8) Conclusions

The experimental and theoretical studies made on this bridge justify the following conclusions:

1. The load distribution on this simple span prestressed concrete slab bridge, as indicated by the actual bending moments, agrees fairly well with the elastic theory. Experimental values are generally lower than theoretical values

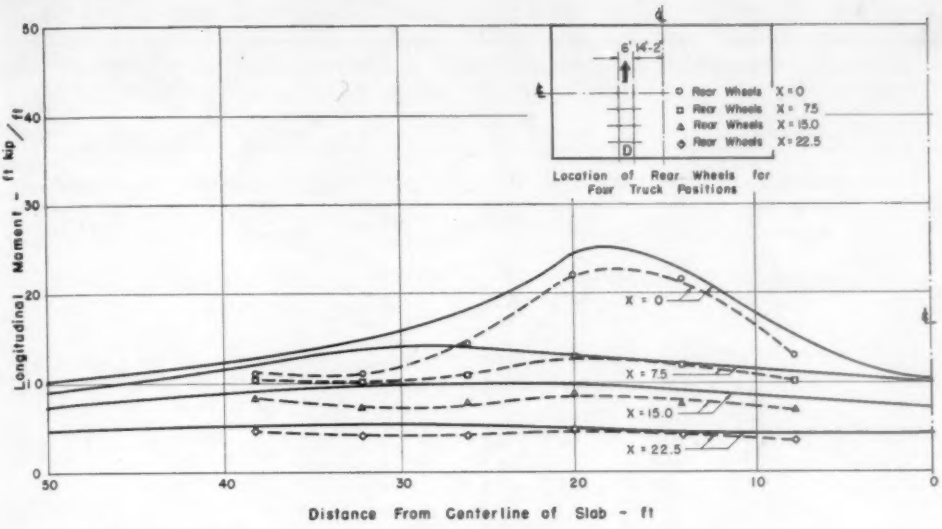

Longitudinal Moment - ft kip / ft
Distance From Centerline of Slab - ft
Location of Rear Wheels for Four Truck Positions
Rear Wheels X = 0
Rear Wheels X = 7.5
Rear Wheels X = 15.0
Rear Wheels X = 22.5
X = 0
X = 7.5
X = 15.0
X = 22.5
D

Fig. 15 - DISTRIBUTION OF LONGITUDINAL MOMENTS ACROSS TRANSVERSE CENTERLINE FOR FOUR TRUCK POSITIONS IN PATH D

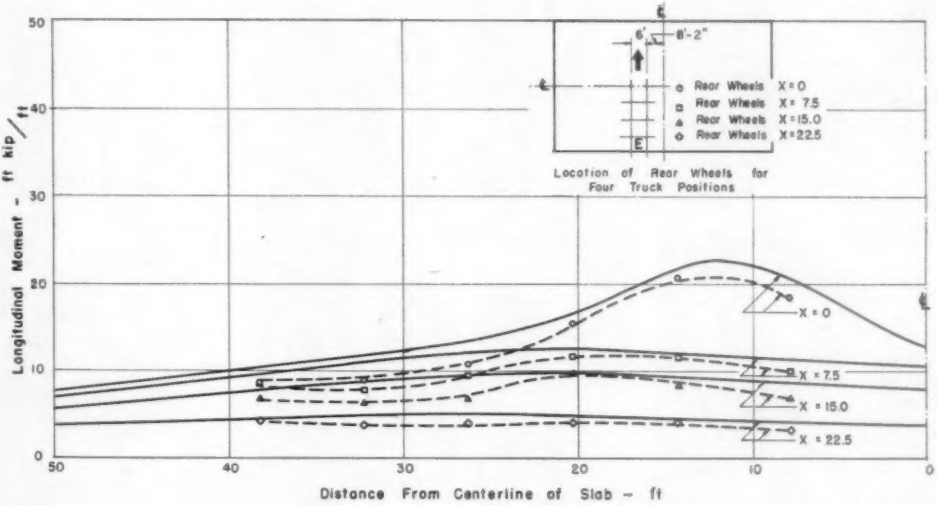

Longitudinal Moment - ft kip / ft
Distance From Centerline of Slab - ft
Location of Rear Wheels for Four Truck Positions
Rear Wheels X = 0
Rear Wheels X = 7.5
Rear Wheels X = 15.0
Rear Wheels X = 22.5
X = 0
X = 7.5
X = 15.0
X = 22.5
E

Fig. 16 - DISTRIBUTION OF LONGITUDINAL MOMENTS ACROSS TRANSVERSE CENTERLINE FOR FOUR TRUCK POSITIONS IN PATH E

with the greatest discrepancy occurring at points near the wheel loads.

2. Transverse prestressing had no measurable effect on the transverse and longitudinal moments produced by wheel loads.

3. Reliable experimental values can be obtained by the use of Carlson gages and with oscillograph recordings, even when the strain readings are relatively small.

4. Since the maximum live load

TABLE 1

STRAINS, STRESSES, AND LONGITUDINAL MOMENTS ACROSS TRANSVERSE CENTERLINE FOR FOUR TRUCK POSITIONS IN PATH A AFTER TRANSVERSE PRESTRESS

Location of Wheels From Centerline of Bridge		Gage Station, Distance from Centerline of Bridge	Longitudinal Strains, millionths	Transverse Strains, millionths	Longitudinal Stresses, psi	Longitudinal Moment, ft. kips/ft.
Rear Wheels	Front Wheels					
		7'-10"	39	9	203	27.6
		14'- 2"	32	6	168	22.8
0	—13 ft.	20'- 2"	23.5	1.0	120	16.4
		26'- 2"	20.5	—	104	14.1
		32'- 2"	15.7	—	80	10.8
		38'- 2"	11.2	—	57	7.7
+ 7.5	— 5.5	7'-10"	25.0	5.3	131	18.5
		14'- 2"	18.7	4.3	98	13.3
		20'- 2"	18.5	2.0	96	12.9
		26'- 2"	18.7	—	95	12.9
		32'- 2"	14.5	—	74	10.0
		38'- 2"	10.5	—	53	7.2
+15.0	—12.0	7'-10"	18.0	3.4	94	12.7
		14'- 2"	13.7	3.0	72	9.8
		20'- 2"	12.7	1.7	66	9.0
		26'- 2"	13.3	—	68	9.2
		32'- 2"	11.0	—	56	7.6
		38'- 2"	7.7	—	38	5.3
+22.5	+ 9.5	7'-10"	8.7	1.5	45	6.2
		14'- 2"	6.8	1.5	36	4.8
		20'- 2"	6.8	1.2	36	4.8
		26'- 2"	7.5	—	38	5.2
		32'- 2"	6.5	—	33	4.5
		38'- 2"	4.5	—	23	3.1

stress produced by the heaviest obtainable experimental truck was no more than 200 psi, while the slab was designed to carry a live load fiber stress of 1320 psi, it was evident that the present empirical rule for the design of slabs of long spans and large width is too conservative from the point of view of stresses. Even the simultaneous loading of all lanes with full impact is not likely to result in stresses as high as those assumed in design. Consequently, a more rational approach to the design would result in considerable saving in prestressing steel, and in a much better behavior of the slab, with less camber and creep.

Acknowledgements

Acknowledgment is given to the California Division of Highways and to Mr. J. W. Jurkovich, Senior Bridge Engineer of the Bridge Department, to the Institute of Transportation and Traffic Engineering, and to Professor T. Y. Lin, University of California, for their help and guidance in the conduct of these investigations.

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Prestressing at Play

by

Walter L. Lowry, Jr.*

Since the theme of our 1959 convention in Miami Beach was "Prestressing at Work", an article on "Prestressing at Play" seems timely and appropriate. This article deals with the use of prestressed concrete for the construction of additional spectator seats in the Clemson Memorial Stadium, Clemson College, Clemson, South Carolina.

Some twenty year ago, the stadium was constructed in a natural bowl-shaped depression and consisted of 45 rows of seats on each side of the playing field extending from end zone to end zone. The favorable terrain made it possible to construct the 45 rows of seats by shaping the clay soil and placing concrete on grade with a relatively small amount of earth work necessary. The rows were constructed along the arcs of concentric circles, the radii varying uniformly from 560 feet for the first or bottom row to 665 feet for the top row. Access to the seats was through eight equally-spaced entrances on grade on each side of the field located at the top or back rows.

At the time the stadium was constructed, the seating capacity of 20,500 was considered adequate but in recent years there has been a demand for seats far in excess of the number available. In 1957, it was decided that an enlargement of the stadium was justified from the standpoint of spectator demand as well as being economically feasible. At a meeting of

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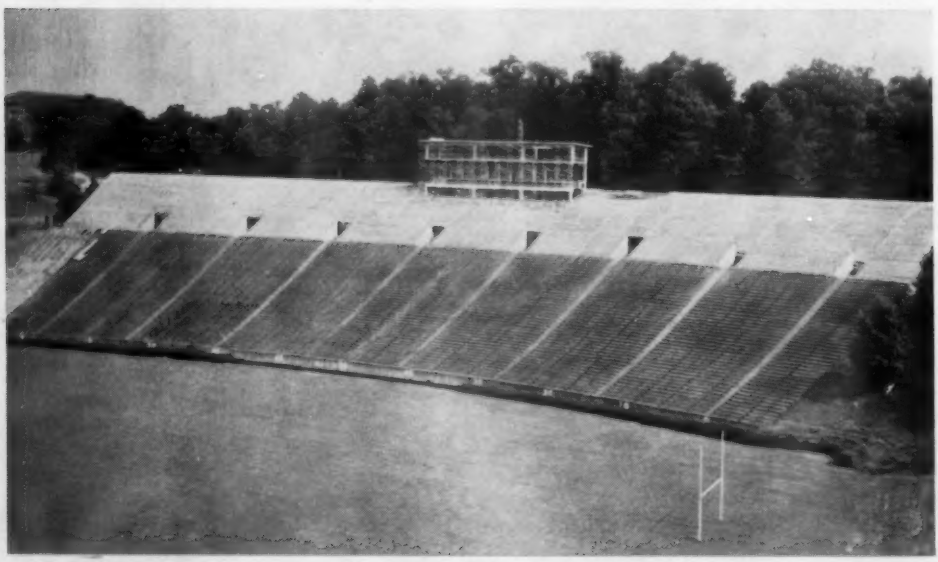


the Athletic Council and the Administration of the College, general approval was given to an overall plan of adding spectator seats, the replacement of the press box by a larger and better equipped one, and the demolition of toilet facilities and concession stands and the construction of new and more adequate ones. A limit of $300,000 was set for carrying out the plan. It was realized that the most economical use of materials and labor was essential and that future maintenance costs must also be considered. A third consideration was the blending of the addition and the original seating from an appearance view point.

Several structural systems were considered and analyzed by the writer and, after sufficient preliminary designs and calculations were made and compared as to original and maintenance costs, the system described below was selected. Since the seating in the original stadium consisted of only 45 rows and since the bottom row of seats was fairly close to the playing field, it was decided that the best arrangement for the additional seating from a spectator standpoint as well as economics would be the construction of additional rows along both sides of the field from end zone to end zone. These additional rows would start at the existing grade of the 45th row and be erected above grade. By making careful estimates of the cost of all facilities except the seating and subtracting this amount from the total sum available, the amount available for spectator seats was determined.

The next step was the selection of the structural system to provide the maximum number of seats possible for the amount of money available keeping in mind the other criteria outlined above. Several analyses were made and compared and a system of using conventional reinforced concrete frames with prestressed modified double-tee sections spanning the distance between the frames was adopted. Using this system, a break-down of costs indicated that the construction of 26 rows on each side of the field providing over 17,000 additional seats was entirely feasible. The existing eight entrances on each side of the field would be utilized and access to both the old and new parts would be at these points. The utilization of these entrances automatically set the location of a structural frame adjacent to each of the entrances with an intermediate frame bisecting the distance between them. Each frame was designed along an extension to the radius of the original stadium at the point where new construction was to begin. The modified double-tees would then become a series of chords and would vary in length from approximately 24 feet for the first row of the addition to approximately 30 feet for the last row.

In the design of the supporting frames, the width of the beam to receive the prestressed sections was set at 14 inches in order to provide a 4 inch bearing for each end of the double-tees resting on the beam and to provide for a six-inch poured strip between the ends of adjacent double-tees to take care of any variations in the double-tees. Bearing plates were doweled into the supporting beams and into the stems of the double-tees. These bearing plates were welded together at one end and left free at the other to permit a small amount of movement at the free end.

By designing the frames along

the extensions of the radii, it was possible to make all frames except the end ones identical, thereby making possible the maximum reuse of forms. Struts were designed for wind stresses and rigidity in the longitudinal direction of the frames. Lateral struts were used in every other bay only, being omitted in the alternate bays to allow for expansion and contraction. No expansion joints per se were designed in the addition as sufficient allowance was made by leaving one end of each double-tee free and omitting the lateral struts between frames in alternate bays. Two years use of the addition with no noticeable ill effects has confirmed this analysis.

The modified double-tee sections were designed for dead load plus a live load of 100 psf. Since the space allowed for each seat is 18 in. wide by 28 in. deep, this provides for a live load of 350 lbs. per seat, considered adequate for both the static and possible dynamic loads. The amount of tension in the prestressing cables was determined by allowing a maximum tensile stress of 300 psi in the bottom fibers of the concrete under full load. A typical calculation illustrating this procedure is given later in the discussion of an additional extension planned for construction during the coming

spring and summer.

The contract for the addition described above was let in March, 1957. Work was started in mid-March and completed in late August and the stadium filled to capacity in the first home game of the 1958 season.

The total cost of the addition to the stadium consisting of over 17,000 spectator seats, a new press box with ample provision for the working press, radio and television coverage, Western Union facilities, etc, a new President's Box, adequate toilet facilities, concession stands, and ticket booths was within the total of $300,000. This addition has given Clemson an outstanding stadium in all respects and provides 38,000 excellent spectator seats. However, by the summer of 1959 it became evident that a further increase in seating capacity was desirable.

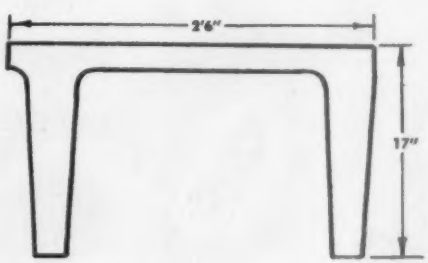
Plans and specifications for the new addition, as was the case in the 1958 addition, have been prepared by members of the college staff and as soon as certain formalities are complied with the invitation for bids will be issued. The new addition will consist of 5700 new spectator seats at the west end of the playing field. The seats will again consist of modified double-tee sections supported on conventional reinforced concrete frames. Fifty rows of seats will be constructed on frames all above natural grade. The new section will be the width of the playing field at the bottom or first row and flair out to approximately 200 feet at the back or 50th row. The elevation of each individual row will be the same as the corresponding row of the existing seating to provide for eventual joining of the new section with the old ones. Under the new section, there will be constructed new dressing rooms, shower and toilet facilities for the home and visiting teams, an officials' dressing room, and toilet facilities and concession stands for the spectators.

Since the new section will be independent from the existing parts, more latitude was possible in the placing of the supporting frames and the resulting spans of the prestressed sections. A total of six frames will be used for the new section, again being placed on an extension to a radius as was done in the 1958 addition. Again the frames will be identical and the spans for the prestressed section will vary from approximately 32 feet for the bottom row to approximately 40 feet at the top. The same procedure as previously used for design will be followed and is illustrated by the sample calculations below for a span of 40 feet.

The dimensions are for a 17 in. deep modified double-tee section as produced by the Ballard-Rice Prestressed Company in Greenville, S. C. by placing longitudinal edge strips in their regular double-tee beds. However, the specifications provide for the substitution of alternate sections of equal capacity upon approval by the College Engineer.

Area = 165 in.^{2}
Wt. = 172 lb. per lin. ft.
I = 4222 in.^{4}

c_b = 11.05″
c_t = 5.95″
S_b = 382 in.^{3}
S_t = 710 in.^{3}



Top fiber stress at center = 473−954+1369 = 888 psi (Comp.)
Bottom fiber stress at center = 473+1772−2545 = 300 psi (Tension)
Bottom fiber stress at end = 473+676 = 1149 psi (Comp.)

$$\text{Camber} = \frac{(1.17)\ (78{,}000)\ (18 \times 3.33 \times 40^2 + 144 \times 5.35 \times 10 \times \frac{2}{3} \times 20)}{(2{,}500{,}000)\ (4222)} = 1.72''$$

$$\text{D.L. Deflection} = \frac{(22.5)\ (172)\ (40^4)}{(2{,}500{,}000)\ (4{,}222)} = 0.94''$$

$\therefore$ Camber under dead load = 1.72″ − 0.94″ = 0.78″

$$\text{L.L. Deflection} = \frac{(22.5)\ (233)\ (40^4)}{(4{,}000{,}000)\ (4{,}222)} = 0.80''$$

$\therefore$ Deflection under live load = 0.80″ − 0.78″ = 0.02″

Three 7/16 in. draped cables are used for the prestressing with eccentricity of 3.33 in. at the end and 8.68 in. at the center.

Moment = (1/8) (233+172) (40^2) (12) = 972,000 in. lb.

$$f_b = \frac{972{,}000}{382} = 2545 \text{ psi}$$

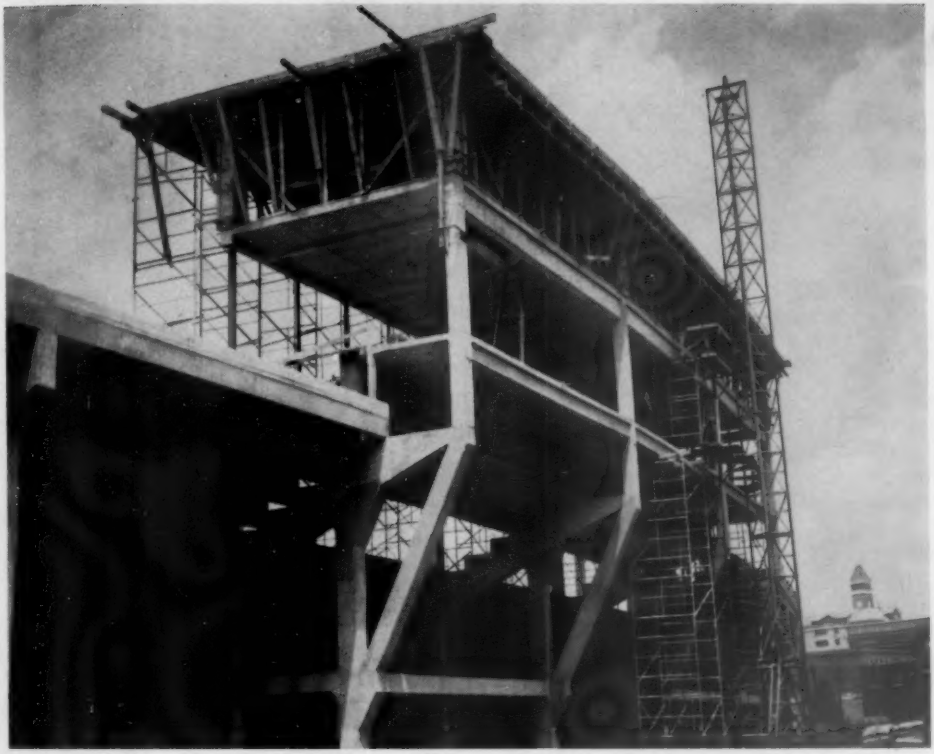
$$f_t = \frac{972{,}000}{710} = 1369 \text{ psi}$$

$$\frac{P}{A} + \frac{Pe}{S_b} = f_b - 300$$

$$P\frac{(382 + 8.68 \times 165)}{165 \times 382} = 2545 - 300 = 2245$$

$$P = 78{,}000 \text{ lb.}$$

Check of fiber stresses:



Our experience at Clemson has convinced us that the use of this type of stadium construction is economical and entirely satisfactory in all respects. Since the construction of the addition in 1958 we have had many visitors from other schools contemplating the building of a new stadium or an addition to an existing one. Without exception, these visitors have been favorably impressed with the utility and appearance, and particularly with the cost figures we have given them.

Research on Shear Strength of Prestressed Beams

by M. A. Sozen*

A Talk presented at the Session on Research and Development 1959 Prestressed Concrete Institute Convention, Miami Beach, Florida

The purpose of this report is to describe the object, scope, and general implications of research at the University of Illinois on the strength under combined bending and shear of prestressed concrete beams. This work is one phase of a major research program on prestressed concrete started in 1951 with the over-all objective of establishing criteria for safe, serviceable, and economical design of highway bridges. Although the goals of the program are practical, the work has been planned to be fundamental and general in scope with the hope that while answers to today's questions are being obtained, the groundwork is being laid for solving some of tomorrow's problems.

To date, the studies on shear strength have involved tests on 181 beams. The types of test specimens are shown in Fig. 1.

The top figure represents the standard simply-supported beam with straight reinforcement subjected to stationary, concentrated loads. The second one from the top is essentially the same type of specimen with the reinforcement draped in the shear spans. For these two types, the ratio of the shear span length, a, to the effective depth, d, varied from 2.4 to 8. The span length was 9 ft. for most of the beams.

The third figure from the top represents a simply-supported beam subjected to a moving load. This type of loading was included in order to project our findings under stationary loads to the analysis of highway bridges.

Tests were also carried out on two-span continuous beams represented by the bottom figure.

The majority of the test beams were pretensioned.

The over-all cross sectional dimensions were 6 by 12 in. for all beams but a few. We had rectangular beams (Fig. 2a), I-beams with 3 and 1-3/4-in. thick webs (Fig. 2b and c), and I-beams with a cast-in-place slab (Fig. 2d).

Ranges of the other major variables were as follows. The longitudinal reinforcement was varied from a minimum of 0.2 to a maximum of 1.0 percent, the effective prestress from zero to 140,000 psi, the concrete strength from a very low value of 1750 to 8600 psi, and the ratio of web reinforcement from 0.1 to 1.35 percent.

Only vertical stirrups were used, the yield point stress of the stirrup steel varying from 30 to 80,000 psi and the spacing from 2.5 to 10 in. The larger dimension corresponds to the effective depth of the beam.

In describing to you some of the results of the tests, it is desirable to start with the observed behavior of

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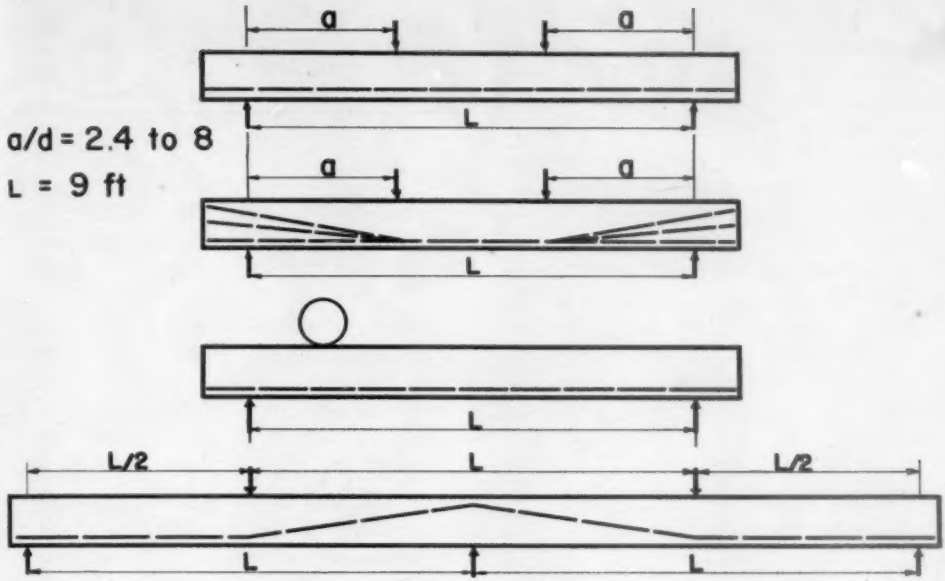


Fig. 1

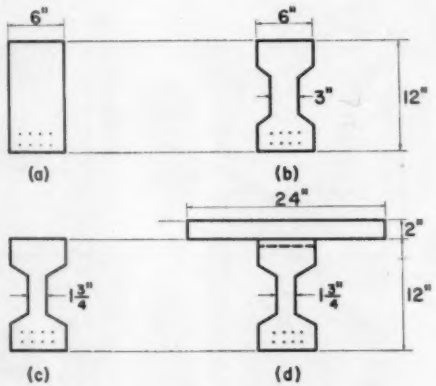


Fig. 2

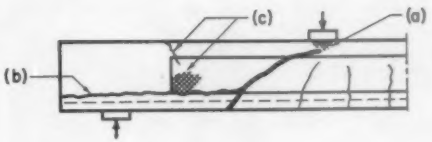


Fig. 3

a prestressed concrete beam without any web reinforcement.

In such beams, the formation of an inclined crack (shown by the thick line in Fig. 3), caused by a combination of bending and shear stresses, represented a very significant stage in the "life" of the beam. The beam carried very little load after the development of the inclined crack; and the increase could not be designated as useful or even reserve strength for all practical purposes.

The failure modes of beams with inclined cracks took several forms. The beams collapsed by:

(1) Concrete crushing in the top flange near the load points as indicated by the letter (a) in Fig. 3.

(2) Separation of the tension flange from the web indicated by (b).

(3) Crushing of the concrete in the web due to arch action indicated by (c).

In all cases, such shear failures resulted in a reduced load carrying capacity in comparison to a flexural failure.

Figure 4 presents a particular comparison of the behavior of I-beams failing in shear and in flexure. These load-deflection curves are based on tests of two almost identical I-beams.

Curve 1 refers to an I-beam which failed in shear almost immediately after inclined cracking and curve 2 refers to an I-beam which had enough web reinforcement to produce a flexural failure. Curve 2 represents an increase of about 100 percent in load and 200 percent in deflection compared to Curve 1. There is no question about the desirability of some form of web reinforcement whenever the likelihood of a shear failure exists.

Figure 5 shows the crack pattern in an I-beam with vertical stirrups, indicated by the dashed lines, failing in flexure. The introduction of web reinforcement did not affect critically the formation of inclined cracks, but it did hinder and could eliminate their drastic consequences.

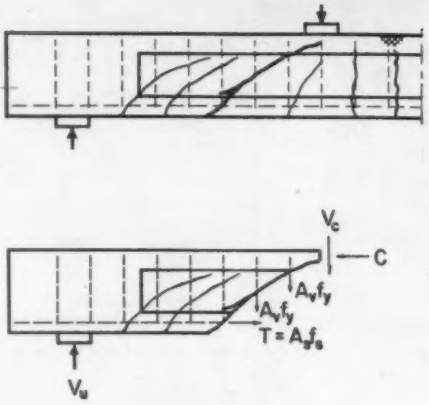


Fig. 5

The results of the tests showed that the amount of web reinforce-

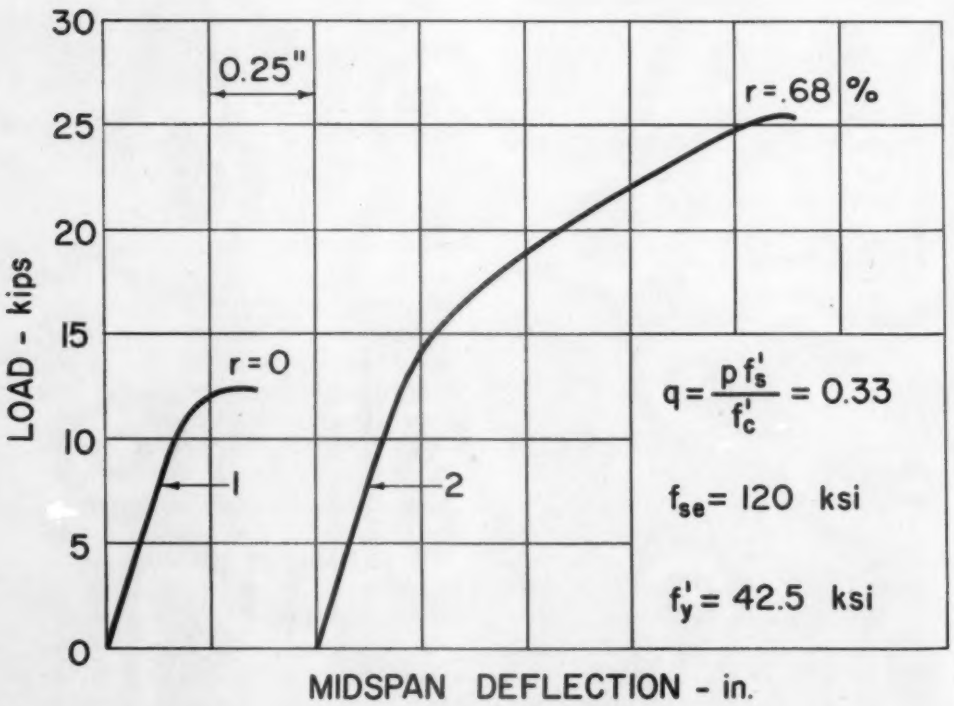


Fig. 4

ment required to produce a flexural failure was proportional to the difference between the shear corresponding to a flexural failure and the shear corresponding to inclined cracking.

A simplified interpretation of the action of web reinforcement can be developed in reference to the free body diagram in Fig. 5. If we assume that the beam can carry the inclined cracking shear, V_c, without any help from the stirrups, the equilibrium of the free body diagram demands that the ultimate shear, V_u, be equal to V_c plus the sum of the forces in the stirrups crossed by the inclined crack, each stirrup working at its yield stress (Maximum yield stress for the stirrups used in the tests was 80,000 psi).

The analysis of the test results according to this concept yielded the expression

$$V_u = V_c + \frac{5}{4}\frac{d}{s}A_v f_{vy}$$

where V_u = ultimate shear
V_c = shear at inclined tension cracking
d = effective depth of beam
s = stirrup spacing
A_v = cross sectional area of one stirrup
f_{vy} = yield stress of the stirrup steel

Figure 6 illustrates an application of this concept to the design of web reinforcement in a beam subjected to a moving load. The maximum shear at any section is that corresponding to the various positions of the load that would produce flexural failure when it acts at midspan. This is the solid line designated V_u. The dead load is ignored. The inclined cracking shear at each section is indicated by the broken curve designated V_c; the tests showed that this quantity decreases as the distance from the reaction increases.

The amount of shear to be carried by the web reinforcement is indicated by the shaded portion. It would, of course, be prudent to start placing stirrups a distance "d" away from the point where none are theoretically required. This arrangement of web reinforcement is diametrically opposite to what would be obtained using common design procedures.

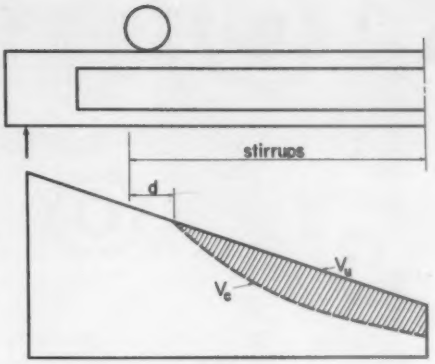


Fig. 6

Figure 7 shows a simply-supported beam at failure. The failure load was reached by successive passes of a simulated moving load going from left to right, the load being increased after each pass. The stirrups in this beam were designed according to the concept discussed above. Their locations are indicated by the vertical broken lines. More stirrups were placed near the midspan than near the reactions.

This beam developed its full flexural capacity. The cracks are shown by the black solid lines on the beam. At failure, there were no inclined cracks near the reactions. Consequently, no stirrups were required at the ends, at least not for strength.

The beam shown in Fig. 8 was loaded with a single stationary load. No stirrups were placed over a large portion of the longer shear span, because no inclined cracks were expected. None appeared. The beam

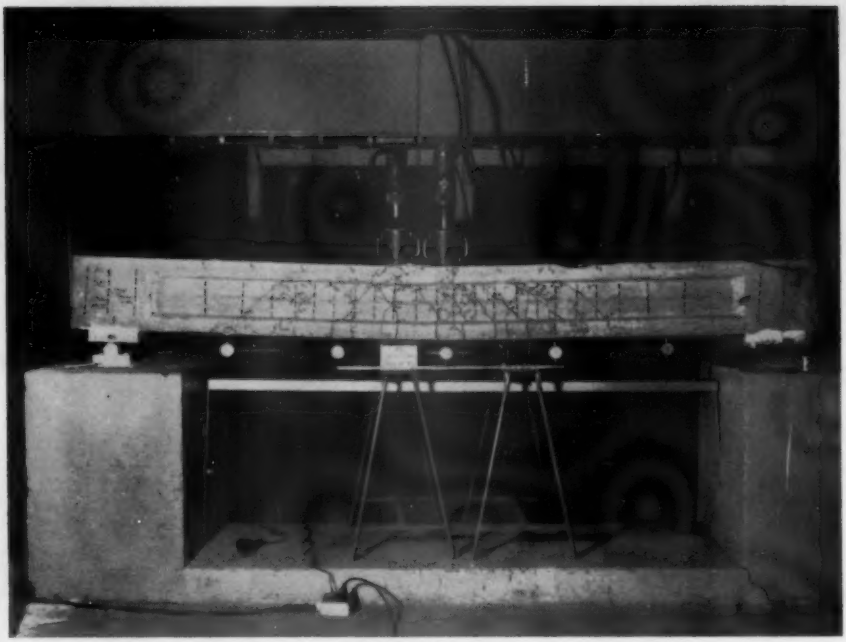


Fig. 7

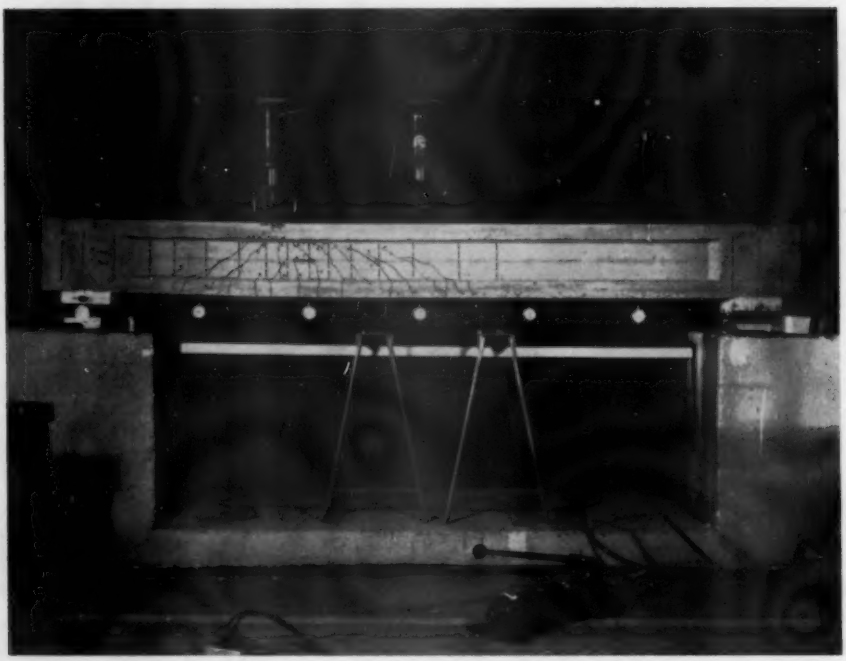


Fig. 8

failed in flexure because, where inclined cracks did appear, they were restrained by the stirrups.

Another interesting aspect of the tests was the comparative behavior of beams with straight and draped reinforcement. The profile of the steel in the beams with draped reinforcement is shown in the upper left-hand corner of Fig. 9. In some of the beams only part of the steel was draped. The two load-deflection curves in this figure present a particular comparison of the effect of draping. They are based on tests of two beams having similar properties except for the shape of the reinforcement profile.

The first curve refers to a beam with straight wires which failed in shear at about 80 percent of its flexural capacity. The second curve refers to a beam with a high drape. The longitudinal steel made an angle of 10° with the horizontal in the shear spans. It also failed in shear. The beam with the straight wires carried about 40 percent more load in comparison to the beam with draped wires.

In the ordinary ranges of the variables, this trend was consistent. There was very little decrease in the load-carrying capacity up to a drape angle of about 2°. Beyond this, there was appreciable decrease in strength as the drape was increased.

This phenomenon has been ascribed to the close relation between a flexural and an inclined crack. Draping the steel reduces the flexural cracking load in the shear span which, in turn, reduces the effective section and invites the inclined crack earlier in the life of the beam.

The investigation of the strength

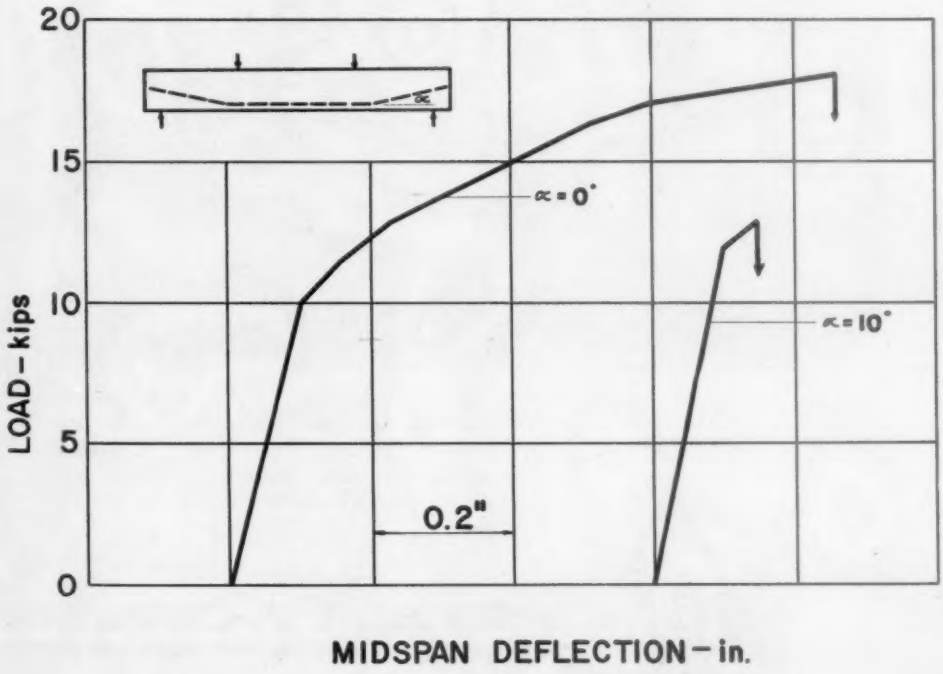
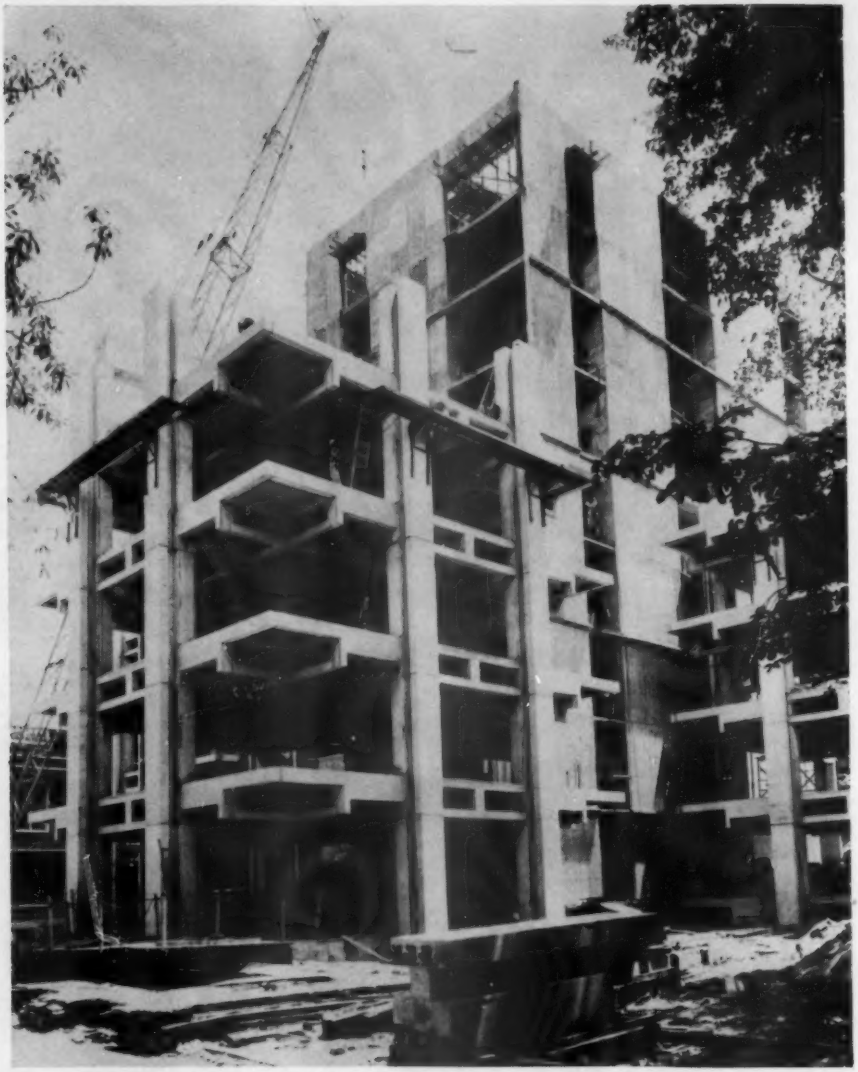


Fig. 9



Credits: Architect: Louis I. Kahn, F.A.I.A.; Structural Engineers: Keast & Hood; Consulting Structural Engineer: Dr. August E. Komendant; Prefabricator: Altantic Prestressed Concrete Co.; General Contractor: Joseph R. Farrell, Inc. Photos: Stressteel Corp.

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Construction men position Stressteel bars in girder prior to post tensioning.

THE THREE 8-story prestressed towers of the new University of Pennsylvania Medical Research Building mark a significant mile-post in the field of precast concrete building construction. These towers abut a cast-in-place central structure. Concrete elements consisted of precast, prestressed trusses, girders and columns. The entire structure is tied together vertically and horizontally by post-tensioned Stressteel bars.

Close tolerances, smooth surface finish, and precision fitting of the pieces were achieved. Using slumps which never exceeded 3 inches, Plastiment provided the necessary high degree of workability and uniform quality in both precast and cast-in-place concrete.

Where slumps of the lowest possible order are required while maintaining practical degrees of workability, Plastiment is the answer. For full details, contact your local Sika representative or write for Bulletin PCD.

ICAL CORPORATION

Passaic, N. J.

of continuous beams is in its exploratory stages. A preliminary series of tests has confirmed that the principles of limit design can be utilized in the design of prestressed beams provided shear failure is prevented. Figure 10 illustrates the ductility of a test specimen at failure. However, continuous beams are also susceptible to shear failures; and the failure is more violent and destructive than that in a simple beam. (Fig. 11)

In conclusion it should be pointed out that after nearly a decade of labor, all the answers are still not known. At the moment, a method of design for shear can be proposed that is safe and reasonably economical. These studies are being continued and in the future, it may be possible to develop a method that is as economical as it is safe.

Acknowledgements

The research program described is a part of the Illinois Cooperative Highway Research Program Project IHR-10, "Investigation of Prestressed Reinforced Concrete for Highway Bridges," carried out by the Civil Engineering Department of the University of Illinois in cooperation with the Illinois Division of Highways and the U. S. Department of Commerce, Bureau of Public Roads. The tests and studies were conducted by the following members and former members of the Civil Engineering Department Structural Research Staff: R. N. Bruce, K. A. Faulkes, C. J. Fleming, N. M. Hawkins, G. Hernandez, L. Hognestad, T. Lum, J. G. MacGregor, and E. M. Zwoyer. The supervision of the work was provided by C. P. Siess and M. A. Sozen.

Fig. 10

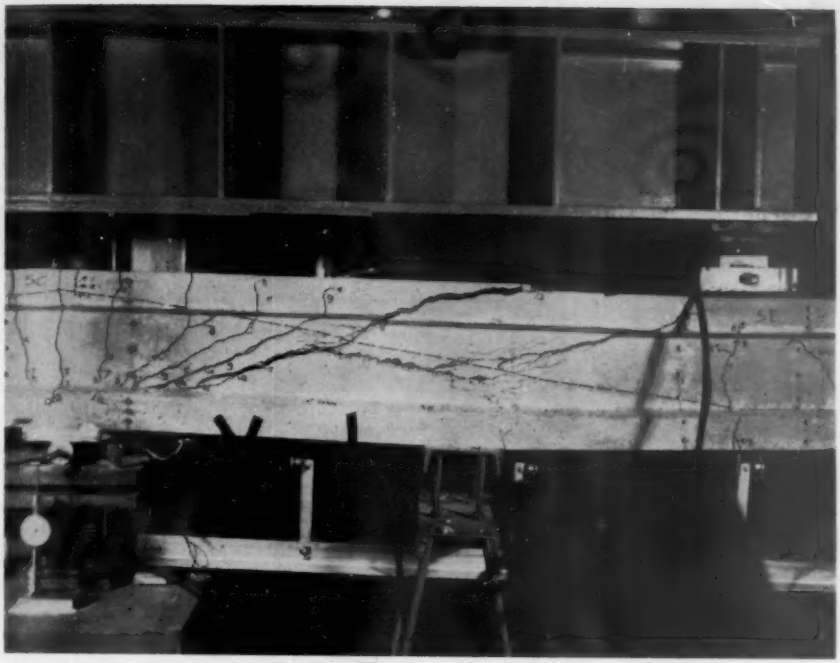


Fig. 11

A Panorama of Prestressing in the United States

by **James D. Piper***

Presented at the
Prestressed Concrete Institute
Convention
Miami Beach, Florida
November 4, 1959

One of the most perplexing phenomena on the American economic scene is the prestressed concrete industry—perplexing, that is, to economists. Never before had they experienced such rapid acceptance of a major innovation in design and construction techniques. Upon closer look, however, prestressing can be seen to have had a sound foundation of research, development and field trials before the first flush of popularity in the United States.

It is fairly well recognized now that the theory of prestressing antedates the present century; but it wasn't until the advent of high tensile strength steel and high strength concretes that prestressing could really come into its own.

It was first used in this country in the construction of circular tanks and pipe 37 years ago. Prestressed tanks similar to the one shown in Fig. 1 have been built in the United States since 1935. The chief engineer of a city which has had such a tank since 1936 has stated, "Not a penny has been spent for maintenance or repairs in twenty years." Linear prestressing, however, was not attempted in this country until 1942 when some pretensioned 29-ft. long channels and I-section members were made for an Anheuser-Busch bottling plant in St. Louis, Missouri.

Fig. 1

(Figure 2) A warehouse floor in Chicago was post-tensioned in 1947 by John A. Roebling's Sons Corp. This three-inch thick slab-on-grade floor was prestressed in two directions using 9/16-in. galvanized 7-wire strand. In the ensuing 12 years, the floor has been subjected to severe loading conditions, but it is still showing no signs of wear.

(Figure 3) The break-through in America came in 1949 with the construction of the Walnut Lane Bridge in Philadelphia, Pa. It was only recently that the U. S. record set by its 160-ft. center span was exceeded. The impact that it made upon our construction industry can be

*Vice President for Promotion, Portland Cement Association.

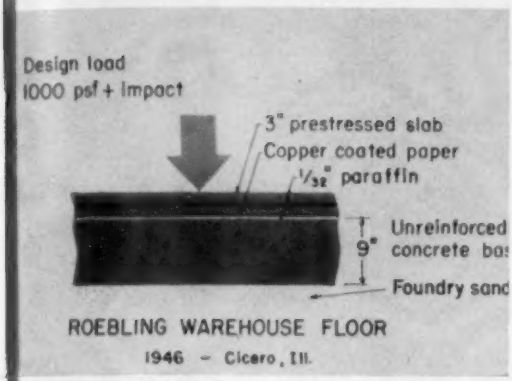


Fig. 2

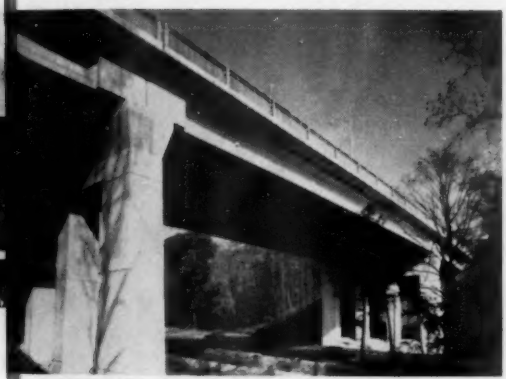


Fig. 3

readily appreciated by reviewing the progress prestressing has made in gaining recognition in the United States since then.

During the doldrums of the interim period between our pioneer efforts and the Walnut Lane Bridge, Europeans had made considerable progress. Several reasons conspired to create a more favorable environment in Europe for growth of the budding industry than here.

For one thing, the critical shortage of building materials overseas during and following World War II encouraged experimentation in means of improving and extending the applications of available materials. Since metals were classified as strategic materials, concrete was the natural answer to almost all building needs. Fortunately, European designers evolved new techniques that were much more than mere stop-gap expedients. One of their most important developments was the crystallization of the concept of prestressing into a workable, practical method of construction.

Another factor which fostered prestressing in Europe may be attributed to the different architect-engineer-contractor relationship which exists there. It is not uncommon for one European firm to assume the responsibility for both the design and construction of a structure. Such an arrangement provided the financial incentive needed for the sound development of the concept of prestressing.

In the United States, initial acceptance was slow because the efficient use of materials characteristic of prestressing could not be capitalized upon until means had been devised to reduce labor costs. These steps to minimize labor mainly took the form of developing production-line techniques for the prefabrication of prestressed members.

Linear prestressing first made its weight felt in the bridge market where the increased spans and reduced depth-to span ratios rendered it a potent competitor for rival materials.

Ever since the Walnut Lane bridge jolted the market out of its complacency, the number of prestressed bridges built each year has soared upward. Today, prestressed bridges are fast becoming a part of the landscape in practically every state of the union. Three girder types have emerged from the early experimenting as the most commonly used in this country—pretensioned box beams, pretensioned I-beams and post-tensioned long-span gird-

ers. The first two types are usually precast to speed erection time, whereas the long-span girders are cast at the job site to reduce transportation charges. In addition to these precast units, the post-tensioned cast-in-place bridge has continued to develop to a point of considerable refinement and sophistication of design.

The state of Pennsylvania leads this country in number of precast-prestressed box beam bridges. Among the many advantages of this type are speed of erection, shallow deck providing maximum headroom and an efficient structural design.

(Figure 4) Less than two days were needed to set in place the deck beams for this bridge and to post-tension them together laterally with high strength bars. Box beams of spans in excess of 100 ft. have been built by pretensioning deflected strands.

(Figure 5) Sites requiring skewed bridges are accommodated easily with prestressed I-beams. Cast-in-place decks can be formed to fit any desired plan. Skews have been accommodated by prestressed beams ranging from this small amount to over 45 degrees. Intermediate or end diaphragms can be used or omitted dependent on design or specification requirements. Continuity can be achieved by erecting simple spans and providing continuous reinforcing steel in the slab over the supports.

(Figure 6) Although standard girder shapes give some measure of uniformity to American pretensioned bridge design, post-tensioned girders are custom designed for virtually every application. Post-tensioning is ordinarily reserved for girders in excess of 100 ft. It finds its greatest use in long-span bridg-

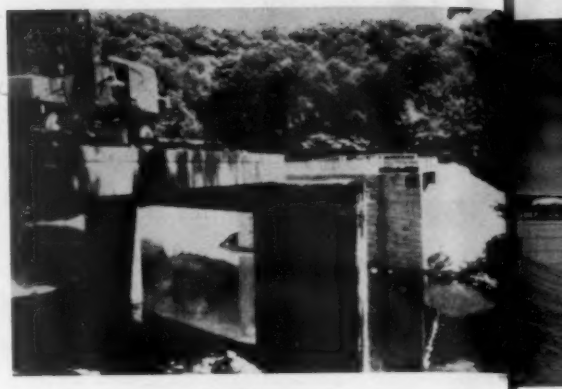


Fig. 4

Fig. 5

Fig. 6

Fig. 7

Fig. 8

es, gymnasiums and auditoriums.

These girders for a bridge over the Milwaukee River in Milwaukee, Wis. were post-tensioned; but they conform to AASHO-PCI girder shape requirements. It points up the versatility of standardized girder shapes.

(Figure 7) Probably the most popular single prestressed girder shape in the United States is the I-beam. In this unusual structure on the Illinois tollroad, pretensioned I-beams made possible a restaurant and shops located over the pavement. Although most I-beams are pretensioned, some have been constructed by post-tensioning or by a combination of pretensioning and post-tensioning. Economics is usually the determining factor and, therefore, this choice is generally at the option of the contractor. Cast-in-place decks are made composite with the beams to produce a structure with excellent riding qualities. Most states have adopted either the AASHO-PCI standards or state standards which closely parallel them. These standard shapes have done much to lower the costs of both design and fabrication. In addition, many prestressed I-beam girders have been manufactured recently without end blocks, as on the Illinois tollroad, resulting in appreciably reduced casting costs.

(Figure 8) One of the most spectacular examples of prestressed bridges in the United States is now under construction in upper New York State. It is the pair of bridges over Oneida Lake whose center spans of 320 ft. set a record in this country. Each center span consists of three parts: two cantilevered sections of approximately 72 ft. each and a drop-in girder of about 230 ft. After the precast cantilever post-tensioned girders for the end spans had been constructed, the post-tensioned drop-in girders were positioned and the entire assembly post-tensioned together to act as one unit.

More an expansion than a new use is the application of prestressing to the construction of long-span bridges. There have been a number of prestressed bridges built in Europe and Central and South America in the 350 to 400 ft. span range. It has been reported that plans have been completed for the Venezuelan government for a prestressed concrete suspension bridge over Lake Maracaibo that will have a central span of 1,312 ft. The 72-ft. wide bridge will provide two two-lane roadways on either side of

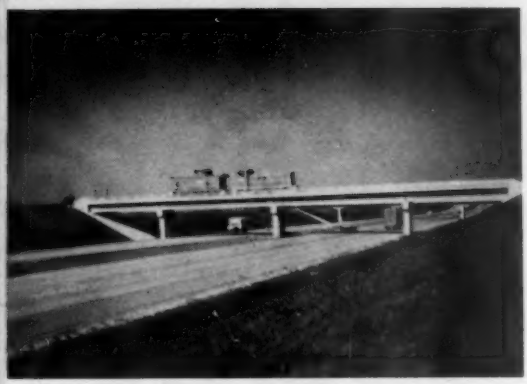


Fig. 9

a single railway track. As was mentioned, the longest American prestressed span is the 320-ft. center span of the Oneida Lake bridge. It represents an increase of 100 ft. over the previous U. S. record holder.

(Figure 9) The first prestressed railway bridge in the United States was a 19-ft. span built in March 1954. American railroads, characteristically conservative in embracing new construction ideas, didn't try another prestressed bridge until 1957. In the last two years, however, a number have been built with spans up to 72 ft. and with the heaviest loadings in the books. This 320-ft. long prestressed bridge carries the Great Southwestern Railroad over the Dallas-Ft. Worth Turnpike. Its two 67-ft. center spans accommodate the divided six-lane highway below.

Although bridges ushered in the era of U. S. prestressing, buildings claim one-half of the total production today, and undoubtedly will account for even more in the future. Stemmed units, such as double-tees, channels, single-tees and single wing tees, constitute the bread-and-butter business of most prestressing plants. The fact that they can be mass produced in fixed forms means that they can compete successfully with floor and roof units long thought to be the lowest cost means of construction.

(Figure 10) A typical example of effective use of double-tee units is this multi-story commercial building in Charlotte, N. C. Typical bays for this type of building are about 30 ft. by 50 ft. with double-tees spanning longitudinally. In construction of this type, prestressed units often compete on first cost alone with practically any type of roofing and flooring.

(Figure 11) Extruded rectangular floor and roof slabs, both cored and solid, are rapidly gaining favor because they are simple to erect and are low-priced, thanks to their ease of manufacture. Several methods of extrusion are used. In this unique system, slabs are extruded on moving pallets which pass through a machine equipped with mandrels, concrete hoppers and vibrators. One extrusion process employs a moving 'mold' which casts the slabs on stationary pretensioning beds. Extrusion offers many advantages—labor is minimized, formwork is eliminated and high-quality, uniform products are assured.

Fig. 10

Fig. 11

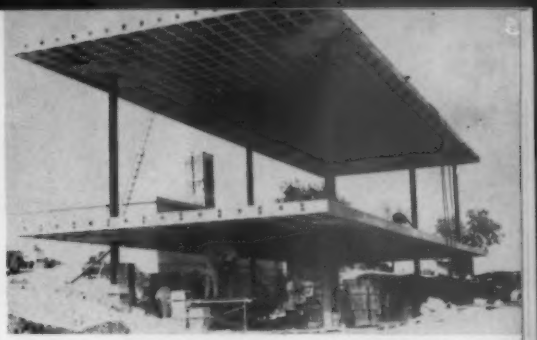


Fig. 13

(Figure 12) Structural beams for buildings have not been standardized to the extent of stemmed slabs but many plants offer a set of typical girders to meet commonly encountered needs. Shelf beams are often used to reduce the total depth of beam and slab. Continuity may be obtained by conventional reinforcing steel in a cast-in-place section in the upper portion of the beams over the supports. This serves to simplify the connection detail and to tie the structure together.

(Figure 13) Lift-slab, a technique native to the United States, also finds great advantages in prestressing. In lift slab work, roof and floor slabs are stack cast on the ground floor and then hydraulically jacked up the columns to the desired height. By post-tensioning the slabs after they have cured, it is possible to achieve control of deflection. Continuity is achieved by properly draping the prestressing steel.

(Figure 14) Prestressing has proven beneficial in shell roof construction, especially folded plates. The steel can be used in the shell itself, as in this folded plate roof; or in domes and cylindrical shells, it can be embedded in the edge beam. Engineers can, as a result, design larger shells and architects can be provided with the slimmer edge beams they often prefer.

Although bridge girders and building members make up the bulk of the prestressing market today, there are a number of other applications.

(Figure 15) Use of prestressed piling has enjoyed almost universal acceptance in the last few years. More than half a million linear feet

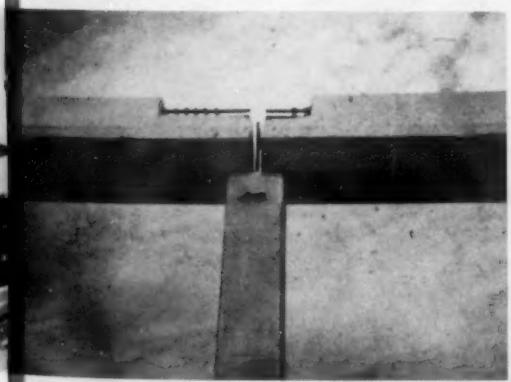


Fig. 12

Fig. 14

Fig. 15

of 21-in. octagonal prestressed piling was used on a single project at Charleston, South Carolina. Average length of the piles was 100 feet. Casting was conducted on a 24-hour cycle. Some projects have used prestressed sheet piling, but development in this area has been restricted largely to marine structures.

As has been stated, one of the earliest applications of prestressing was in the construction of tanks. A novel means of positioning and tensioning the prestressing steel for circular tanks has been developed. After the walls of the tank have been cast and cured, a self-propelled machine hung from the top of the walls winds the strand around the tank. (Fig. 1) With this machine it is possible to wind in two days all the steel needed for a one million gallon tank. As it is wound, it is stressed to 140,000 psi. Afterwards, a thincoating of pneumatically placed mortar is applied. The thin walls and ease of construction achieved with prestressed tanks render them highly competitive.

Among other structures for which prestressing has been used with success are poles and towers, retaining walls and swimming pools. Casting operations are now underway in Florida for the production of 276 prestressed transmission line towers. These 110-ft. high giants will carry power lines 63 miles.

Naturally, the first prestressed designs in the United States leaned heavily on foreign techniques and reflected a conservativeness typical of pioneer efforts with new engineering concepts. The massive end blocks, heavy diaphragms and transverse prestressing on 14-ft. centers of the Walnut Lane Bridge are mute testimony to this fact. This bridge marked the start of an era by effectively demonstrating a principle. Since that time American engineers, many of them in this audience, have been tailoring the principle to our economics to render it practical for us, as well as workable. Since the U. S. construction scene is markedly different in many respects from that in Europe, it was only logical that several new techniques should be developed.

One of the most important of these developments was the use of 7-wire strand in pretensioned work, in place of the small diameter wire commonly used in Europe. It made pretensioning of large members economical. Starting cautiously with ¼ inch strand, the size was gradually increased through 5/16; 3/8 and 7/16 to a ½-in. strand. Each increase in size was preceded by research and field tests on the nature of bond in pretensioned members subjected to static and repeated loading. Perfecting and popularizing long-line pretensioning—made

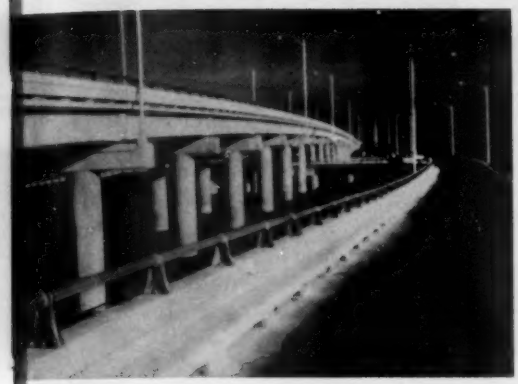


Fig. 16

Fig. 17

possible by 7-wire strand—has rendered mass production of prestressed members a reality in the United States.

(Figure 16) Lake Pontchartrain Bridge forcibly brought home to U. S. engineers the fact that an entire bridge span could be precast easily and economically in one unit. It illustrated dramatically that production-line fabrication of precast-prestressed members had arrived. As Construction Methods and Equipment magazine put it, *World's Longest Highway Bridge—A Mass-Produced Marvel.* Since then the marveling has been replaced by the realization that mass-produced prestressed members are here to stay.

By and large, bridge members of spans up to 100 ft. have been standardized. Of course, this does not obviate the prospect of new or refined designs; but it does suggest that work in this field has been stabilized.

(Figure 17) In contrast, the design of building components still shows indications of being under development. Even the primacy of the popular double-tee unit has been challenged.

Typical early stemmed members were 14-in. double-tee units. The size of double-tees was increased gradually up to 18 inches and even deeper. Today several plants are able to produce single-tees that span up to 120 feet. Shown is an experimental beam which combines some of the characteristics of a box girder and a stemmed unit.

A major advantage of prestressing shared by all its applications is the use of high-strength concretes. It was only a short time ago that 3,000 psi concrete was considered a high-strength mix. Prestressing makes full use of 5,000 and 6,000 psi concrete.

This helps increase the strength-to-weight ratio of concrete appreciably—always a desirable attribute in terms of competitive bidding. Reinforced concrete in the United States has a strength-to-weight ratio approximately one-third that of steel. However, with the 5,000 psi concretes commonly used in prestressing, a ratio of about one-half that of steel can be obtained. Mixes achieving 8,000 psi would narrow steel's strength-to-weight ratio margin over concrete to less than one-quarter. Now that high strength, lightweight aggregate concretes have been developed, it is possible to predict a ratio almost equaling

that of structural steel for concrete in the foreseeable future. That this is readily apparent to the competitors of prestressed concrete is attested to in the activities underway in an ASTM committee to raise the yield point of structural grade steel.

Even the great strides prestressing has taken in the last few years represent merely the beginning of the infiltration that it will undoubtedly make into many market areas. The manufacture of many prestressed products is now one of mass production. Its use is spreading in ever widening circles to many more areas. Crystal ball gazing is not one of my fortes but I think we can obtain some fairly reliable glimpses of the future of prestressing simply by examining the experimentation of today.

(Figure 18) One of these fields is prestressed pavements. Although prestressed highways seem far away, at this stage, its use in the construction of heavily-loaded airport pavement gives promise of moving out of the experimental stage in the near future. A taxiway at Biggs Air Force Base near El Paso, Texas, was recently constructed of three different types of slabs: 24-in. unreinforced slabs; 19-in. reinforced slabs; and 9-in. prestressed slabs.

Detailed data on the cost and performance of the pavement types will be obtained. This taxiway might well presage an era of prestressed concrete airport pavements to meet the more rigorous needs of jet aircraft.

(Figure 19) A new industry on the verge of full-fledged production is prestressed concrete railway ties. The Association of American Railroads, in conjunction with PCA and others, has conducted extensive tests on numerous prestressed ties. A prototype tie derived from the experience of European railways and the results of these tests has been designed. Plans have already been finalized with the Atlantic Coast Line and Seaboard Air Line railways to install test sections of the tie. One firm in Florida is designing a machine with which it will be possible to mass produce the tie to price

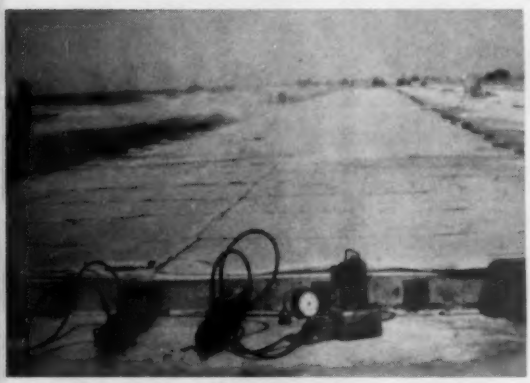


Fig. 18

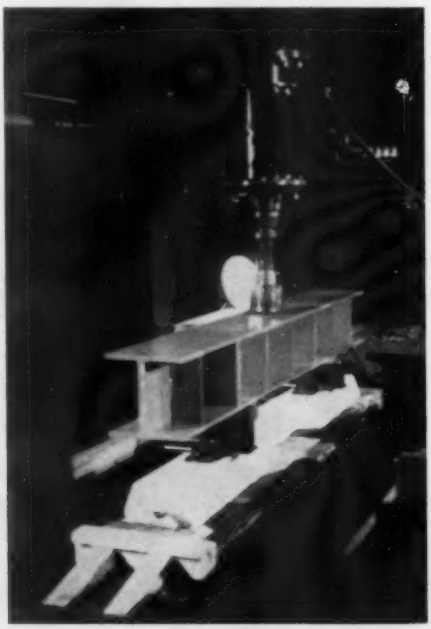


Fig. 19

it competitively with wood ties. Although prestressed ties will cost somewhat more individually than those of wood, the fact that they can replace wood ties two-for-three and permit the use of smaller tie-plates may render them strong contenders in this new market area.

(Figure 20) Prestressing of decorative wall panels is a field largely untouched as yet. However, prestressing offers a number of advantages when applied to precast wall panels. Panel thickness can be reduced to a bare minimum without encountering handling difficulties or high breakage rates. Insulative materials can occupy a greater proportion of the panel cross-section.

Fig. 20

(Figure 21) The huge market of home construction is another virgin territory as far as prestressing is concerned. This is a model of a prestressed home whose prototype was easily assembled of precast-prestressed components. Naturally, the components for multi-unit construction of such homes would have to be of a size and weight that could be handled by relatively light equipment. The advent of lightweight aggregates capable of producing high strength concretes will be a boon to prestressing in this, as well as other market areas.

Fig. 21

(Figure 22) Studies are underway which are aimed at originating simple means of achieving continuity in prestressed structures—most especially those with pretensioned members. Once such details are ironed out, prestressing will receive a tremendous boost in competitive power since it will be possible to span even greater distances, reduce depths and carry greater loads for less cost.

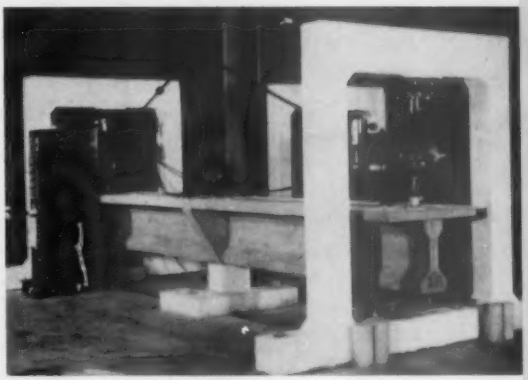


Fig. 22

(Figure 23) Sound and imaginative engineering, open-mindedness on the part of all the members of

Fig. 23

Fig. 24

the construction team, hard work on the part of many far-sighted individuals, pioneering work by equipment and materials suppliers—all have contributed to the inestimable success enjoyed by prestressing in its early years in the United States.

For prestressed building components to capture as large a share of the market as they should, a building code must be promulgated. In America, the important job of formulating a recommended practice was entrusted to a group of more than 100 leaders in the prestressing industry. Next step on this front is to secure incorporation of the recommended practice in code language into building codes throughout the nation. This is now being done by the ACI through cooperation with your organization.

(Figure 24) If the promising future of prestressed concrete is to be fully realized, however, it will be necessary to continue and expand present activity in research and development. The Portland Cement Association recently put into operation two new laboratory buildings which will enable us to step-up our work in these areas. It is now possible for us to conduct fire tests on full-scale prestressed beams in the furnace in our Fire Research Center. This will help to remove a barrier in the path of prestressing. Our new Structural Laboratory contains facilities for static and dynamic load testing of full-scale prestressed beams.

The formation of the Prestressed Concrete Institute has stimulated interest in prestressing greatly. The numerous tests already sponsored by PCI and the plans being made by the committee for fire test planning will undoubtedly foster even greater acceptance of prestressing.

Many other groups are currently conducting research on various aspects of prestressed concrete construction. Several of the larger colleges and universities are engaged in research projects in this area: the University of Florida on torsion; the University of Illinois on shear and bending; Lehigh University on fatigue and shear; and the University of California on two way slabs and columns. The U. S. Air Force is constructing prestressed airport runways to assess their practicality. Many privately owned plants are carrying on or supporting developmental and research projects.

The enthusiasm evidenced on all hands for this new approach to construction has set up a momentum that has amazed all on the American construction scene. Its past has been eye-opening. But the wonder of prestressing is that its future seems brighter yet.

Average Concrete Stress Along Prestressing Steel in Prestressed Concrete Beams

by Kenneth Kai-I Diao*

I INTRODUCTION

In estimating the loss of prestress due to elastic shortening and creep effect of concrete in prestressed concrete beams, it is necessary to calculate the average direct stress in concrete at the level of the prestressing steel. For straight, prismatic, simple beams with undraped pretensioning strands, the average concrete stress along the c.g.s. due to a uniformly distributed load over the whole span is simply 2/3 of the value at the mid-span. However, for haunched beams, continuous spans, cantilevers, and beams with curved prestressing units, the evaluation of such stresses may become quite cumbersome. In this article, rational yet simple formulas for computing the average direct stress in concrete along parabolic post-tensioning tendons in simply supported beams are derived. The underlying principle of this procedure may be readily applied to other more complicated cases. Similar formulas for computing the average direct stress in concrete along the c.g.s. of straight strands are also presented for the ready use in engineering offices.

II NOTATION

A = cross sectional area (gross) of beam, sq. in.

e_o = eccentricity of curved tendon axis at mid-span with respect to centroidal axis of beam section (gross), inches.

e = eccentricity of curved tendon axis at any point with respect to centroidal axis of beam section (gross), inches.

e_s = eccentricity of c.g.s. of straight pretensioning strands with respect to centroidal axis of beam section (gross), inches.

f = direct stress in concrete at level of centroid of curved post-tensioning tendon at any section, psi.

f_{cs} = average direct stress in concrete for whole span along c.g.s. of curved tendons or c.g.s. of straight strands, psi; positive for compression and negative for tension.

F_s = effective prestress of pretensioning strands, lbs.

F_t = effective prestress of post-tensioning tendons, lbs.

I = gross moment of inertia of beam section with respect to centroidal axis, $(in)^4$.

K = ratio of distance between load point and left end of beam to span length, dimensionless.

L = span length of beam, inches.

*Structural Engineer, Madigan-Hyland Consulting Engineers, New York, N. Y.

M = bending moment at any section along beam, in-lbs.
M_o = bending moment at mid-span for a parabolic moment curve, in-lbs.
M_p = bending moment at load point for single concentrated load, in-lbs.
P = concentrated load, lbs.
r = center sag of parabolic post-tensioning tendon, inches.
w = uniformly distributed load, lbs. per inch.
x, y = coordinates of any point on parabolic axis of post-tensioning tendon with origin at its center, inches.
x_a, y_a = coordinates of any point on parabolic axis of post-tensioning tendon with origin at its left end, inches.

III DERIVATION OF EQUATIONS FOR PARABOLIC TENDONS

In the following derivation of f_{cs}, theoretically, the cross sectional area should include the transformed steel area, the moment of inertia should be computed on the basis of the transformed section, and the eccentricities of tendons and strands should be measured from the centroidal axes respectively of the transformed section and net section. However, in practice for the sake of simplicity the gross concrete section is always used in computing all these items. The error induced is usually insignificant.

For ordinary cases, the difference between the inclined prestress at the end of a parabolic tendon and its horizontal component is small and therefore is to be ignored in deriving Eq. (9) below.

(1) Uniformly Distributed Load

In Fig. 1, let us assume the axis of the post-tensioning tendon in a simple supported prestressed concrete beam to follow a parabolic curve of second degree such as,

$$y = \frac{4r}{L^2} x^2$$

with the origin at the center of the curve.

Then $$e = y - e_o$$

and $$e = \frac{4r}{L^2} x^2 - e_o \qquad (1)$$

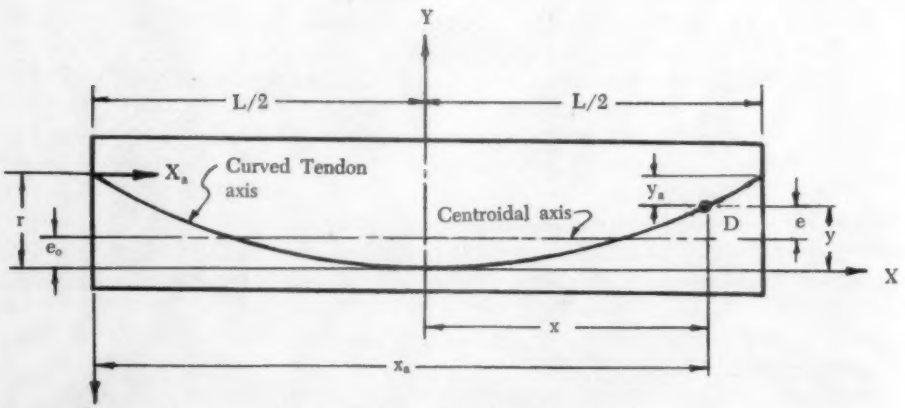


Fig. 1 Profile of Curved Tendon

Since the moment curve due to a uniformly distributed load w is a parabola as shown in Fig. 2, the moment at any point D can be expressed in terms of the mid-ordinate M_o, i.e.

$$M = M_o\left(1 - \frac{4x^2}{L_2}\right) \tag{2}$$

By the usual elastic theory, the corresponding direct stress in concrete at the level of the post-tensioning tendon is

$$f = Me/I \tag{3}$$

The average value of the direct stress in concrete for the whole span along the curved tendon axis is

$$f_{cs} = \frac{1}{L}\int_{-L/2}^{L/2} f\,dx$$

By substitution of Eqs. (1), (2) and (3), it yields

$$f_{cs} = \frac{1}{L}\int_{-L/2}^{L/2} \frac{M_o}{I}\left(1 - \frac{4x^2}{L^2}\right)\left(\frac{4r}{L^2}x^2 - e_o\right)dx$$

Performing the integration from $x = -L/2$ to $x = L/2$ for a prismatic beam, we have

$$f_{cs} = \frac{2M_o}{15I}(r - 5e_o) \tag{4}$$

or

$$f_{cs} = \frac{wL^2}{60I}(r - 5e_o) \tag{4a}$$

(2) Straight Pretensioning Strands

The effect of straight pretensioning strands with an eccentricity of e_s

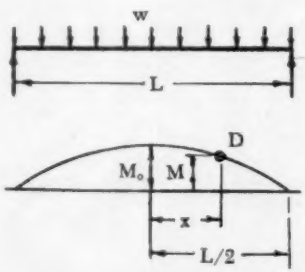


Fig. 2 Moment Diagram for a Uniformly Distributed Load

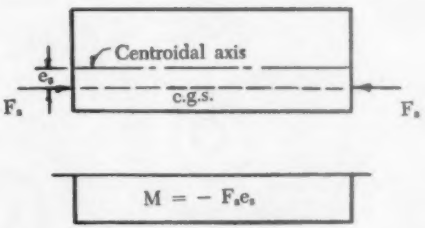


Fig. 3 Moment Diagram for an Eccentric Pretension

on a prestressed concrete beam can be replaced by two components: a moment $(-F_s e_s)$ and a concentric prestress F_s acting at the centroid. The moment is uniform throughout the span, as shown in Fig. 3. The negative sign here indicates that, when the c.g.s. of the strands is located below the centroidal axis, the moment will produce tension in the top fiber of the beam.

By applying the same procedure as discussed under the preceding heading, it follows that the average direct stress in concrete along the curved tendon axis due to the moment $(-F_s e_s)$ is

$$f_{cs} = \frac{1}{L}\int_{L/2}^{L/2} \left(\frac{-F_s e_s}{I}\right)\left(\frac{4r}{L^2}x^2 - e_o\right) dx$$

Upon integration as indicated, it becomes

$$f_{cs} = -\frac{F_s e_s}{3I}(r - 3e_o) \tag{5}$$

Then, the total average direct stress in concrete along the curved tendon axis due to the eccentric pretensioning strands is

$$f_{cs} = \frac{F_s}{A} - \frac{F_s e_s}{3I}(r - 3e_o) \tag{6}$$

(3) Draped Post-tensioning Tendon

In Fig. 4, the moment curve due to a parabolic post-tensioning tendon can be resolved into two parts: namely, a rectangle with the height of

$$M = F_t(r - e_o) \tag{7}$$

and a parabolic with the mid-ordinate of

$$M_o = -F_t r \tag{8}$$

By following Eq. (5) and Eq. (4), the average concrete stress along the curved tendon axis respectively due to the rectangular moment $F_t(r - e_o)$ and the parabolic moment $(-F_t r)$ is given by

$$f_{cs} = \frac{F_t(r - e_o)}{3I}(r - 3e_o)$$

and

$$f_{cs} = -\frac{2F_t r}{15I}(r - 5e_o)$$

Then, the total average concrete stress along the curved tendon axis due to the post-tensioning force F_t becomes

$$f_{cs} = \frac{F_t}{A} + \frac{F_t(r - e_o)}{3I}(r - 3e_o) - \frac{2F_t r}{15I}(r - 5e_o) \tag{9}$$

(4) Concentrated Loads

The moment diagram for a single concentrated load P acting at a distance KL from the left end of the beam is shown in Fig. 5. To simplify the integration, the left end of the curved tendon axis is selected as the

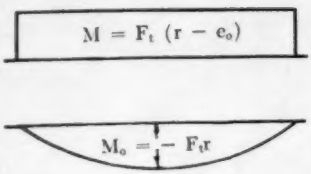


Fig. 4 Component Moment Diagrams for a Curved Post-tension

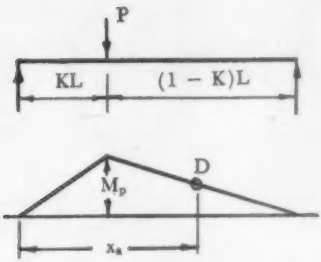


Fig. 5 Moment Diagram for a Concentrated Load

origin of the coordinates. (Fig. 1) The equation for the parabol is given by

$$y_a = \frac{4r}{L^2}(Lx_a - x_a^2)$$

then
$$e = (r - e_o) - \frac{4r}{L^2}(Lx_a - x_a^2)$$

For $O \leqq x_a \leqq KL$, $\quad M = M_p x_a / KL$

For $KL \leqq x_a \leqq L$, $\quad M = M_p (L - x_a)/(1 - K)L$

Thus,
$$f_{cs} = \frac{1}{L}\int_0^{KL} \frac{M_p x_a}{KLI}\left[(r - e_o) - \frac{4r}{L_2}(Lx_a - x_a^2)\right] dx_a$$

$$+ \frac{1}{L}\int_{KL}^{L} \frac{M_p(L - x_a)}{(1 - K)LI}\left[(r - e_o) - \frac{4r}{L_2}(Lx_a - x_a^2)\right] dx_a$$

After carrying out the integration, we have the expression for the average direct stress in concrete along the curved tendon axis due to a concentrated load P at KL from the left end of the beam,

$$f_{cs} = \frac{M_p}{6I}\left[(1 - 2K + 2K^2)r - 3e_o\right] \tag{10}$$

As a special case, when P acts at the center of the span, i.e., K = 0.5, Eq. (10) becomes

$$f_{cs} = \frac{M_p}{12I}(r - 6e_o) \tag{11}$$

or
$$f_{cs} = \frac{PL}{48I}(r - 6e_o) \tag{11a}$$

For multiple concentrated loads, the combined f_{cs} may be obtained by superimposing the individual values of f_{cs} for each concentrated load. For example, when two equal concentrated loads act at the third-points of the beam, M_p for one concentrated load $P = 2PL/9$, $K = 1/3$, and $(1 - 2K + 2K^2) = 5/9$. Then, from Eq. (10)

$$f_{cs} = 2 \times \left[\frac{2PL}{9} \times \frac{1}{6I} \left(\frac{5r}{9} - 3e_o \right) \right]$$

or

$$f_{cs} = \frac{PL}{243I} (10r - 54e_o) \tag{12}$$

Similarly, for three equal concentrated loads at the quarter-points of the beam,

$$f_{cs} = \frac{PL}{384I} (23r - 120e_o) \tag{13}$$

and for four equal concentrated loads at the fifth-points of the beam,

$$f_{cs} = \frac{PL}{1875I} (146r - 750e_o) \tag{14}$$

IV WORKING FORMULAS

Similar formulas for computing the average concrete stress along the c.g.s. of straight strands may be readily derived by using the same procedure. Working formulas thus obtained for straight strands and those for parabolic tendons as previously derived are summarized here for the convenience of reference.

V ILLUSTRATIVE EXAMPLES

Example 1.

A concrete beam of AASHO-PCI Type IV section and 80-ft simple span is pretensioned with 46-7/16″Φ strands and post-tensioned with a 12 x 0.276″Φ-wire Freyssinet cable. The c.g.s. of the strands is 12 inches above the bottom throughout the beam. The c.g.s. of the cable is 2.75 inches above the bottom at mid-span and 50 inches at both ends, following a parabolic curve. Assume an effective prestress of 695,500 lbs for the strands and 93,000 lbs for the cable. The centroidal axis of the gross section is located at 24.73 inches above the bottom. Compute the average concrete stress along the curved axis of the cable under the combined action of the beam's own weight and the effective prestress.

Solution: AASHO-PCI Type IV, $A = 789$ sq in, $I = 260{,}740$ in^4

$w = 822$ lbs per ft

$e_o = 24.73 - 2.75 = 21.98$ in

$r = 50.00 - 2.75 = 47.25$ in

$e_s = 24.73 - 12.0 = 12.73$ in

For the beam's own weight,

$$M_o = \frac{1}{8} \times \frac{822}{12} \times (80 \times 12)^2 = 7{,}891{,}000 \text{ in-lbs}$$

From Eq. (4),

$$f_{cs} = \frac{2 \times 7{,}891{,}000}{15 \times 260{,}740}(47.25 - 5 \times 21.98) = -253 \text{ psi (tension)}$$

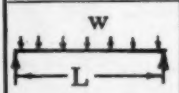
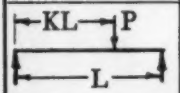
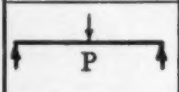
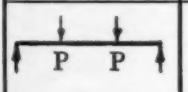
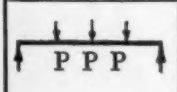
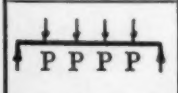
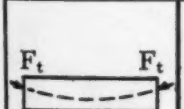
For the straight pretensioning strands,

$$F_s e_s = 695{,}500 \times 12.73 = 8{,}854{,}000 \text{ in-lbs}$$

From Eq. (6),

$$f_{cs} = \frac{695{,}000}{789} - \frac{8{,}854{,}000}{3 \times 260{,}740}(47.25 - 3 \times 21.98)$$

$$= 881 + 212 = 1{,}093 \text{ psi (compression)}$$

Cases	f_{cs} along Straight Strands	f_{cs} along Parabolic Tendons
	$-\frac{2M_o}{3I} e_s$ or $-\frac{wL^2}{12I} e_s$	$\frac{2M_o}{15I}(r - 5e_o)$ or $\frac{wL^2}{60I}(r - 5e_o)$
	$-\frac{M_p}{2I} e_s$	$\frac{M_p}{6I}[(1 - 2K + 2K^2)\, r - 3e_o]$
	$-\frac{M_p}{2I} e_s$ or $-\frac{PL}{8I} e_s$	$\frac{M_p}{12I}(r - 6e_o)$ or $\frac{PL}{48I}(r - 6e_o)$
	$-\frac{2PL}{9I} e_s$	$\frac{PL}{243I}(10r - 54e_o)$
	$-\frac{5PL}{16I} e_s$	$\frac{PL}{384I}(23r - 120e_o)$
	$-\frac{10PL}{25I} e_s$	$\frac{PL}{1875I}(146r - 750e_o)$
	$\frac{F_s}{A} - \frac{F_s e_s}{I} e_s$	$\frac{F_s}{A} - \frac{F_s e_s}{3I}(r - 3e_o)$
	$\frac{F_t}{A} - \frac{F_t e_s}{3I}(r - 3e_o)$	$\frac{F_t}{A} + \frac{F_t(r - e_o)}{3I}(r - 3e_o) - \frac{2F_t r}{15I}(r - 5e_o)$

Note: All concentrated loads in the above table are equally spaced except as otherwise shown.

For the draped post-tensioning cable,

$$F_t(r - e_o) = 93{,}000\,(47.25 - 21.98) = 2{,}350{,}000 \text{ in-lbs}$$
$$F_t r = 93{,}000 \times 47.25 = 4{,}394{,}000 \text{ in-lbs}$$

From Eq. (9),

$$f_{cs} = \frac{93{,}000}{789} + \frac{2{,}350{,}000}{3 \times 260{,}740}(47.25 - 3 \times 21.98) - \frac{2 \times 4{,}394{,}000}{15 \times 260{,}740}(47.25 - 5 \times 21.98)$$
$$= 118 - 56 + 141 = 203 \text{ psi (compression)}$$

Thus, the combined average concrete stress along the curved axis of the post-tensioning cable is

$$f_{cs} = -253 + 1{,}093 + 203 = 1{,}043 \text{ psi (compression)} \qquad \text{Ans.}$$

Example 2.

A concrete beam of 12 × 18 rectangular cross section and 32-ft simple span is post-tensioned with an effective prestress of 120 kips. The prestressing unit is in the shape of a parabolic curve with an eccentricity of 5 inches below the centroidal axis at mid-span and a center sag of 6 inches. The beam supports a permanent concentrated load of 10 kips at the center. Find the average concrete stress along the curved axis of the prestressing steel.

Solution: $A = 12 \times 18 = 216$ sq in, $I = 12 \times 13^3/12 = 5{,}832$ in^4,
$w = 216 \times 150/12 \times 12 \times 12 = 18.75$ lbs per in,
$P = 10{,}000$ lbs, $e_o = 5$ in, $r = 6$ in,
$F_t = 120{,}000$ lbs, $L = 32 \times 12 = .384$ in

For the weight of beam, from Eq. (4a)

$$f_{cs} = \frac{18.75 \times (384)^2}{60 \times 5{,}832}(6 - 5 \times 5) = -150 \text{ psi (tension)}$$

For the post-tension,

$$F_t(r - e_o) = 120{,}000 \times (6 - 5) = 120{,}000 \text{ in-lbs}$$
$$F_t r = 120{,}000 \times 6 = 720{,}000 \text{ in-lbs}$$

From Eq. (9),

$$f_{cs} = \frac{120{,}000}{216} + \frac{120{,}000}{3 \times 5{,}832}(5 - 3 \times 6) - \frac{2 \times 720{,}000}{15 \times 5{,}832}(5 - 5 \times 6)$$
$$= 556 - 89 + 412 = 879 \text{ psi (compression)}$$

For the concentrated load, from Eq. (11a),

$$f_{cs} = \frac{10{,}000 \times 384}{48 \times 5{,}432}(6 - 6 \times 5) = -329 \text{ psi (tension)}$$

Thus, the total average concrete stress along the curved axis of the post-tensioning steel is

$$f_{cs} = -150 + 879 - 329 = 400 \text{ psi (compression)} \qquad \text{Ans.}$$

Prestressed Concrete Airport Pavement

by J. P. McIntyre*

A Talk presented at the 1959 Prestressed Concrete Institute Convention, Miami Beach, Florida

Since your last convention, an event has happened that may well rank with Philadelphia's Walnut Lane Bridge in the history of prestressed concrete in the United States. This event I speak of is the first application in this country of prestressed concrete pavement at an active air base.

In June of 1958 the Air Force gave the green light for the construction of a prestressed concrete taxiway at Biggs Air Force Base, Texas. This decision by the Air Force is of great importance to the Prestressed Concrete Institute. It opens up a vast new field for prestressed concrete in this country, a field that has tremendous potentials.

Before I show some pictures of the construction of the Biggs Taxiway, let us ask ourselves two important questions. (1) "What are the advantages of a prestressed concrete airfield pavement over a conventional non-reinforced pavement?"

Savings and Advantages.

1. Stronger pavement to withstand increasing wheel loads.
2. Elimination or close control of cracks.
3. A 95 per cent reduction in number of joints.
4. Reduction in damage to joint materials from jet blasts.
5. Less maintenance.
6. Thinner slab requiring less initial grading.
7. Reduction in quantity of material.
8. Better serviceability with longer service life.

The second question is: "With these advantages what is needed to make prestressed concrete competitive?"

What is needed?

Prestressed industry's experience and know-how to develop:

1. Mass Production Methods—A machine must be developed to position and tension all the strands in a 25-ft. lane simultaneously. Improved methods of creating transverse prestress in the pavement must be developed.

2. Promotion of prestressed pavement with its many advantages and savings.

Please keep these thoughts of mechanization in mind as we look at

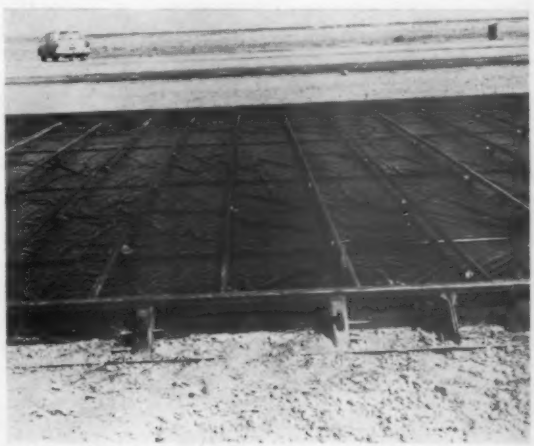


Fig. 1

*Director of Civil Engineering, Headquarters, United States Air Force, Washington, D.C.

some pictures of the construction of the prestressed taxiway at Biggs Air Force Base, Texas.

Figure 1

This prestressed taxiway was designed by the Army Corps of Engineers, Ohio River Division, Rigid Pavement Laboratory. It was designed for post-tensioning both longitudinally and transversely. The thickness is 9-in. and total length is 1500-ft. In this view you see the rigid transverse conduits in place supported by metal chairs. A rolled sand base is covered with polyethylene paper to reduce sliding friction between the slab and the subbase. This is a view of a 25-ft. lane, the total width of the taxiway being 75-ft. or three 25-ft. lanes.

Figure 2

Applying hand tension to straighten flexible longitudinal tendons. Each section of prestressed pavement between expansion joints is 500 ft. with three such sections making the 1500 ft.

Figure 3

Transverse and longitudinal tendons in position just prior to placing of concrete. Each 25-ft. lane contains 13 longitudinal flexible conduits spaced 23-in. center to center, each conduit containing twelve ¼-in. wires. The transverse rigid conduits carrying 6 wires are spaced approximately 27-in. center to center.

Figure 4

Placing of concrete, spreading machine in operation.

Figure 5

Vibrating machine being raised to clear transverse tubes.

Figure 6

The design called for an effective longitudinal prestress of 350 psi and transverse prestress of 175 psi. The wires were anchored at a stress of

Fig. 2

Fig. 3

Fig. 4

Fig. 5

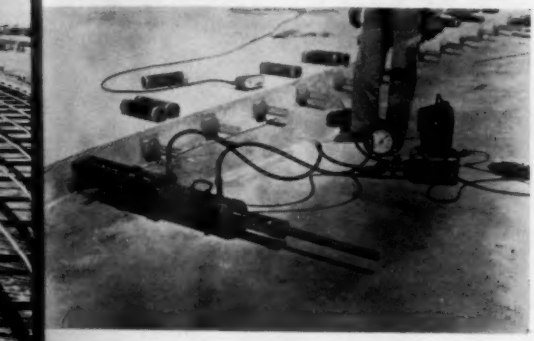


Fig. 6

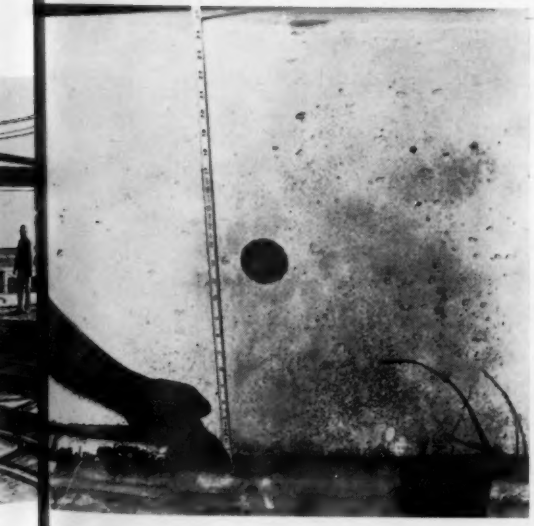


Fig. 7

168,000 psi, to meet these requirements. Note the thickness of the 9-in. pavement.
Figure 7

This is the 24-in. non-reinforced pavement constructed in the same taxiway. This taxiway contains three pavement designs: this 24-in. non-reinforced pavement, a 19-in. bar reinforced pavement, and the 9-in. prestressed pavement. All three pavements were designed to carry heavy bomber traffic with 265,000 lb. gear load.

Now let us look into the future of prestressed airport pavement. This is the Jet Age and new Jet Age airports are urgently needed. A good example of what is required is the Dulles International Airport, Washington, D. C.'s first jet airport.
Figure 8

This airport will have two parallel runways 11,500 ft x 150 ft. and one cross-wind runway 10,000 ft. x 150 ft. The total scope, including runways, taxiways and apron, is 1,196,000 sq. yd. of pavement. The total cost of pavement will be approximately $7,700,000. As you can see, this is no small project.

To study the effect on the prestressing industry of one contract the size of Dulles International Airport, let us assume that the three runways and their respective taxiways were prestressed to 360 psi longitudinally and 200 psi transversely. The amount of 7/16 in. high tensile strand required would be approximately 18,000,000 ft. or 3,400 miles. This is enough strand to stretch from Miami to the Swiss Alps. It is the equivalent of strand required to prestress 2600 large bridge I-beams each 90 ft. long. It would require 1500 large size reels, each holding 12,000 ft. of strand.

The Dulles Airport is already under construction. Others will quick-

Fig. 8

ly follow. Many of our existing airports will be expanded with longer runways and taxiways. The door has been opened and the field is there to be developed.

The time to act is now. Prestressed concrete airfield pavement can be made competitive if mass production methods of applying the required prestress to the pavement are developed. When this is accomplished, I predict that conventional concrete airfield pavement will give way to prestressed pavement like the horse and buggy did for the automobile.

Research on Fatigue and Shear in Prestressed Concrete

by
Robert F. Warner*

A Talk Presented at the Session on Research and Development, 1959 Prestressed Concrete Institute Convention, Miami Beach, Florida

It is a pleasure to describe some of the concrete research work which is being conducted at Lehigh University. At present two main investigations are in progress; one is concerned with the fatigue properties of prestressed concrete flexural members, the other deals with the shearing strength of prestressed concrete beams. In view of the limited time available, the main part of this talk will be devoted to a description of the fatigue program. However, since one of the main purposes of this Research Session is to acquaint everyone with the nature and the extent of research work which is currently being carried out, I want to first describe in brief outline the shear investigation at Lehigh.

SHEAR FAILURE IN CONCRETE BEAMS.

In the shear investigation at Lehigh, theoretical studies have been made of the ultimate strength and of the diagonal cracking load of prestressed concrete members. Shear tests on approximately fifty beams have now been completed, and the results are at present being analyzed on the basis of earlier theoretical work. One of the test series was of especial significance in the manner in which the beams were loaded and the effect of reloading to a point near where an inclined crack had formed previously. These tests were designed to point out important differences which exist between laboratory tests and field conditions.

The final aim of the experimental and theoretical work is to obtain a better understanding of the shear failure mechanism, which can be used to develop safe, and at the same time economic methods for designing transverse web reinforcement.

FATIGUE RESISTANCE OF PRESTRESSED CONCRETE FLEXURAL MEMBERS

I think all of us are familiar, at least to some extent, with the phenomenon of fatigue failure. We know, for example, that a specimen of steel can fail in fatigue when it is subjected to a large number of repetitions of loading, even though each individual load is considerably less than the static ultimate strength of the specimen. It is probable that every construction material is susceptible to fatigue failure and it is not therefore surprising that laboratory tests have indicated that the strength of a prestressed concrete member under repeated loading can be considerably less than the static ultimate strength. This

*Research Associate in Civil Engineering, Lehigh University, Bethlehem, Pa.

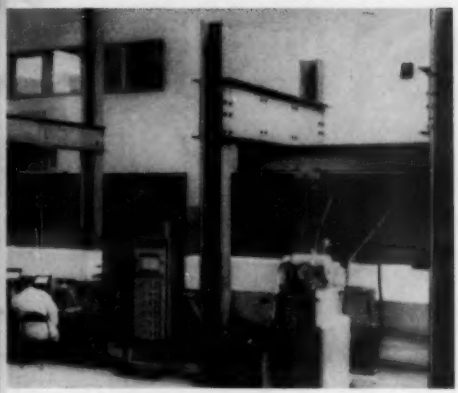


Fig. 1

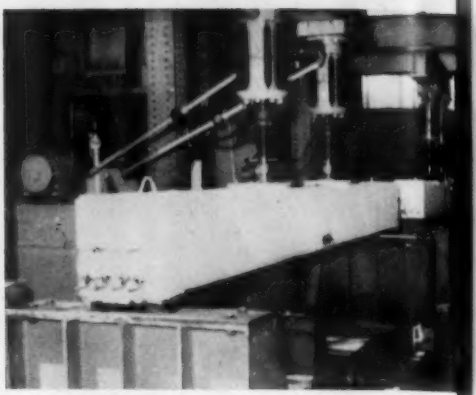


Fig. 2

can be illustrated by reference to a small series of fatigue tests conducted on prestressed concrete railway slabs.

Figures 1 and 2 These tests were carried out on three pairs of beams. One beam in each pair was tested statically while its companion was tested under repeated loading. The specimens were 19 ft. long, 16-1/2 in. wide and 18 in. deep. The pretensioning steel consisted of eleven 1/2 in. dia. strands prestressed to 60 percent of the ultimate stress. As can be seen in these figures two equal point loads were applied at the third points of the beams.

Table 1 The test results are summarized on this table. Column 2 gives the static ultimate strength of the static test beams, A, D and F. In the repeated load cycles applied to beams B, C and E, a minimum load was always maintained to preserve the centering of the loading heads on the beam and the beam on its supports. In all three tests the value of the minimum load was 4 kips per jack. The maximum values of the loading cycles are shown in the next-to-last column, and the number of cycles to failure in the last column.

In each of these tests we can see that failure has been brought about by a cycle of repeated loading in which the maximum load is between 55 and 60 percent of the static ultimate load. The difference in strength, of more than 40 per-

TABLE 1

Beam	Computed Fatigue Strength			Observed Fatigue Strength		
	Min. Load	Max. Load	No. of Cycles	Min. Load	Max. Load	No. of Cycles
B	4.0	33.0	1,000,000	4.0	30.2	1,516,000
C	4.0	32.0	1,000,000	4.0	30.3	970,500
E	4.0	31.4	1,000,000	4.0	28.9	954,000

cent, is quite large and raises a number of pertinent questions concerning the fatigue properties of prestressed concrete members. For example: How do we determine the fatigue resistance of a prestressed concrete beam? What is the factor of safety against failure in fatigue? What is the likelihood of fatigue failure occurring in a member under field conditions?

The fatigue investigation at Lehigh University was initiated in an attempt to find answers to these questions.

Let us now consider the ways in which fatigue failure can occur in a flexural member.

Three modes of failure are possible:

(a) Tensile fatigue failure of the prestressing strand.

(b) Compressive fatigue failure in the upper fibers of the concrete.

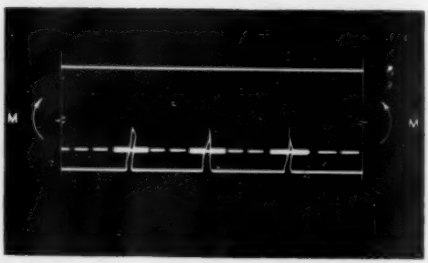


Fig. 3

(c) Progressive bond breakdown between the prestressing strand and the surrounding concrete.

Figure 3 We know that wherever a flexural crack forms there will be an adjacent region in which the bond is broken; this region will extend as the number of load repetitions increases. However, in most members of "normal" dimensions and containing strand reinforcement it is extremely unlikely that these regions of broken bond which surround the adjacent cracks, to cause a general bond breakdown in the beam.

For all practical purposes we can assume that failure will occur either by fatigue in the tension steel or fatigue in the upper fibers of the concrete. It follows that, in order to calculate the fatigue resistance of a member under constant cycles of repeated loading, we need two pieces of information. First, we must know the fatigue properties of the plain concrete and steel, that is, we must know those critical stress ranges which will cause fatigue failure to occur in the materials after a given number of cycles. Second, we must have a means of determining the ranges of stress which are induced in the concrete and steel by a loading applied to the member, or vice versa, of determining the loadings which would produce given stress ranges.

Equations have been developed for the calculation of stresses at cracked and uncracked sections due to flexural loading, and experimental studies have been conducted to determine S-N curves and modified Goodman-Johnson diagrams for steel strand and high strength concrete. With this information it is now possible to estimate the fatigue resistance of flexural members subjected to load cycles of constant magnitude.

Table 2 For example, in this table we compare the computed and observed strengths for the three fatigue test beams described earlier. The fatigue strengths have been calculated for one million repetitions of loading.

In practice however, members are very rarely subjected to the same load cycles, time after time.

TABLE 2

Beam	Static	Fatigue Strength		
	Ultimate Load	Min. Load	Max. Load	No. of Cycles
A	59.9	—	—	—
B	—	4.0	30.2	1,516,100
C	—	4.0	30.3	970,500
D	55.9	—	—	—
E	—	4.0	28.9	954,000
F	53.9	—	—	—

If for example we were to consider a bridge girder, the magnitude of its loadings would vary from cycle to cycle and instead of having one constant load cycle repeated a large number of times we would have varied cycles of repeated loadings. **Figure 4** The question which thus arises, is how to treat the case of a beam with a load frequency distribution such as in this figure.

Current work is centered on this problem and consists of an exten-

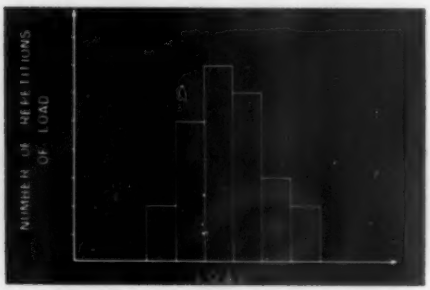


Fig. 4

sion of the experimental study of the fatigue properties of the concrete and steel to deal with varied stress patterns.

Figures 5 and 6 Two sizes of specimens are being used in the concrete program; 2 x 4 in. and 6 x 12 in.

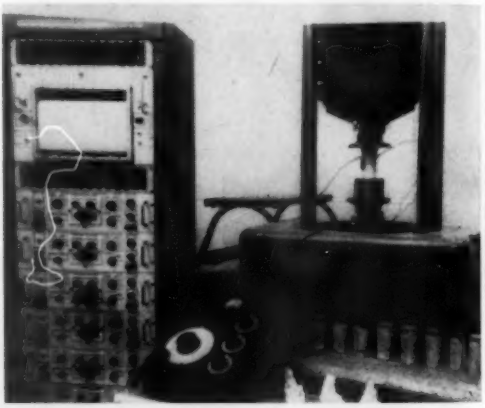


Fig. 5

cylinders. Figures 5 and 6 show two views of a 2 x 4 in. specimen under test in a Vibrophore fatigue testing machine. These tests are designed to provide information on

(a) effect of speed of loading on fatigue strength

(b) scatter to be expected in concrete fatigue tests

(c) variation in the stress-strain relation with number of repetitions of loading

(d) fatigue under varied cycles of stress and cumulative damage in concrete.

When this test program, and a similar test program for steel strand

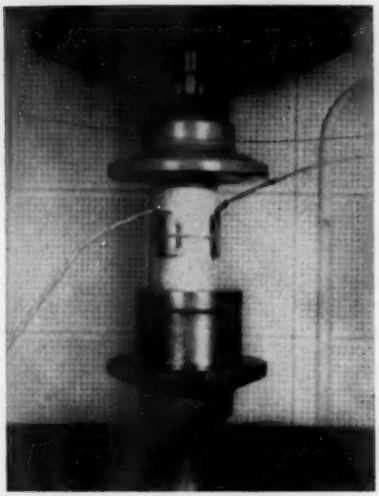


Fig. 6

have been completed, we hope to be able to determine the general fatigue properties of prestressed concrete flexural members and to be able to recommend suitable factors of safety against fatigue failure.

Even at present, however, the theory of fatigue failure developed for constant cycles of repeated loading is useful in a number of particular cases and can be used to estimate the influence of the various parameters on fatigue strength.

For instance it can be shown that the fatigue resistance of a member is considerably improved by the application of prestress. Indeed if the prestress is sufficiently high to prevent the formation of cracks in the member under load, then it is reasonably certain that fatigue failure will not occur. It should be emphasized that although fatigue failure is unlikely to occur at working load in a fully prestressed member, fatigue considerations are important when determining suitable factors of safety. The present research work, dealing with failure under varied repeated loadings, is designed to provide useful data on the question of the factor of safety.

Acknowledgements

The following organizations are sponsoring the concrete research work at Lehigh University: Pennsylvania Department of Highways; Bureau of Public Roads; American Steel and Wire Division, U. S. Steel Corporation; American-Marietta Company, Concrete Products Division; John A. Roebling's Sons Corporation; Reinforced Concrete Research Council.

Tentative Recommended Practices for Grouting Post-tensioned Prestressed Concrete

These Recommended practices were prepared by the Committee on Grouting of Post-tensioned Tendons which consists of the following members:

Meyers Van Buren, Chairman	Raymond International
Ernst Gruenwald, Consultant	Lone Star Cement Corp.
A. F. DiStasio	Intercontinental Equipment Corp.
A. L. Elliott	Division of Highways, California
H. J. Godfrey	John A. Roebling's Sons Corp.
Tadius J. Gutt	George Rackle and Sons Company
Walter H. Larson	Stressteel Corporation
Howard J. McGinnis	Portland Cement Association
James M. Polatty	Corps of Engineers
M. Schupack	Schupack and Zollman

This technical committee has submitted this report to the PCI Technical Activities Committee. TAC has made its review and has in turn submitted these Recommended Practices to the Board of Directors for approval. Comments and suggestions on these Recommended Practices are invited. All discussions should be directed to the Prestressed Concrete Institute, 205 West Wacker Drive, Chicago 6, Ill. The discussions will be reviewed by the Committee and some will be published in the Readers Comment section of this JOURNAL. Discussion on these Recommended Practices will be closed January 1, 1961. Reprints of this report are available from PCI headquarters for 10¢ per copy.

101—Scope

These recommendations cover the grouting of post-tensioning units of prestressed concrete members.

The purpose of grouting is to provide permanent protection to the post-tensioning steel and to develop the designed ultimate strength of the structural member.

102—Definition of Terms

1. **Admixture**—Any material added to the grout other than portland cement, water, natural or manufactured sand.
2. **Cored Duct**—A duct which has been formed in the concrete member by a material which has been entirely removed prior to grouting.
3. **Duct**—The hole or void provided in the concrete for the post-tensioning unit.
4. **Grout**—A fluid mixture of cement and water with or without the addition of sand or admixtures.
5. **Grout Hole**—An inlet or outlet for grout, water or air.
6. **Neat Cement Grout**—A grout without sand.

7. **Post-Tensioning**—The method of prestressing concrete in which the tensioning unit is placed under stress after the concrete has reached a specified strength and the reactive force of the stressed tensioning unit is resisted by the concrete.

8. **Post-Tensioning Unit**—The prestress reinforcing whether wires, bars, or strands used single or in groups.

9. **Tendons**—Another name for post-tensioning units.

10. **Vent Hole**—Same as grout hole.

103—Materials

103.1 Portland Cement

Portland cement should conform to one of the following:

Specifications for portland cement .....ASTM C150
Specifications for air-entraining portland cement .....ASTM C175
Specifications for portland blast furnace slag cement .....ASTM C205
Specifications for portland-pozzolan cement..ASTM C340

103.2 Sand

The sand should conform to that section of specifications for Concrete Aggregates—ASTM C33 referring to Fine Aggregates except that paragraph 3—grading should not apply.

103.3 Water

The water used in the grout should be potable, clean and free of injurious quantities of substances known to be harmful to portland cement, aggregate or prestressing steel.

103.4 Admixtures

Admixtures should not be used until shown by service record or by tests to have no harmful effect on prestressing steel or concrete. Admixtures containing chlorides or nitrates should not be used. Aluminum powder of the proper fineness and quantity or other approved materials should be added to obtain a maximum of 10% expansion of the grout when measured unconfined. If fly ash is used it should conform to ASTM C350. If pozzolonic material is used it should conform to ASTM C402.

104 Ducts

104.1 Forming

(a) **Cored Ducts**—Cored ducts should be formed with no constrictions which would tend to block the passage of grout. All coring material should be removed.

(b) **Formed Ducts**—Ducts formed by sheath left in place should be of a type that would not permit the entrance of cement paste. They should retain shape under the weight of the concrete. The material used in the sheath should not cause electrolytic action or decompose and it should transfer stresses as required. Metallic sheaths should be of a ferrous metal.

104.2 Grout Holes

All ducts should have grout holes at both ends. For draped cables all high points should have a grout hole, except where cable curvature is small, such as in continuous slabs. For ducts which have reverse curves, either horizontal or vertical, and which contain tendons made up of a number of parallel units, of either strand or wires, consideration should be given to the desirability of placing a grout hole near each tangent point.

104.3 Duct Size

For cable type units (tendons made up of parallel wires or strands) duct area should be 60% larger than the cross sectional area of the tendon. Where parallel wires are definitely grouped and held in a definite position for the full length of the tendon, this does not apply.

For bar units or large individual strands the inside diameter of the duct should be a minimum of ¼" larger than the diameter of the tendon. For certain construction schemes practical tolerance requirements may require considerably larger ducts.

105 Equipment

105.1 Mixing

(a) The mixing equipment should be of a type which would produce uniform thoroughly blended grout with a maximum of air entrapment.

(b) Accessory equipment such as scales and liquid measures should be provided to accurately batch all materials.

(c) The mixer should be of such capacity that in combination with the pump hopper a complete duct can be grouted without interruption.

105.2 Injection

(a) A pump should be used to provide the pressure and volume required for grout injection. Injection of grout by compressed air (pressure pot) is not recommended.

(b) The hoses, valves and fittings should be water tight and of sufficient strength and capacity to handle the pressure and volume required.

105.3 General

All equipment should be kept clean and in good working order.

106 Mixing

(a) Water should be added to the mixer first, followed by portland cement, fly ash, admixture and sand as required.

(b) Mixing should be of such duration as to obtain a uniform thoroughly blended grout, without excessive temperature increase.

(c) No water may be added to the grout to increase the flowability which has been decreased by delayed use of the grout.

(d) It is essential that the water content of the grout should be kept as low as possible. The water content should generally be less than 5½ gallons per bag of cement.

107 Placing

(a) The duct should be cleaned of all foreign material prior to grouting. All ducts should be flushed with water or blown out with air.

(b) All grout should be screened. Injection should be continuous from one end and should continue until grout of a consistency equivalent to that injected should flow from all vent openings. The vent openings should be progressively closed in the direction of flow.

(c) In the event of a blockage or a disruption of grouting, all grout should be removed from the duct by flushing the duct with water or air. It is advisable that a dependable high pressure water or air supply be available when grouting.

(d) After the entire duct is filled, all vents and openings should be closed and the grouting pressure at the injection end should be raised to a minimum of 100 psi and held for a minimum of 10 seconds.

(e) Upon removal of the injection

equipment after (d) has been completed, the injection hole should be plugged to prevent the loss of grout.

(f) Agitation should be required in the pump hopper if bleeding is evident.

108.2 Compressive Strength

The grout should have a minimum compressive strength at 7 days of 2,500 psi. Specimen should be cured in the molds approximately 24 hours, then stored and tested according to ASTM Designation C-109. Specimens should be molded in 2″ cubes.

108.3 Trail Mixes

Prior to grouting the trial mixes should be made to determine a mix which will meet the requirements 108.1 and 108.2 for the anticipated grouting temperatures.

109–Cold Weather

109.1 Storage

Ducts should be kept free of water to avoid damage due to freezing.

109.2 Concrete Temperature

At the time of grouting the temperature of the concrete should be 45° F or higher and maintained at this temperature until job cured 2″ cubes reach a minimum compressive strength of 800 psi.

108 Testing

108.1 Plastic Volume Change

(a) There should be no shrinkage of the grout while in the plastic state.

(b) Grout for the volume change determination should be taken directly from the mixer immediately after completion of mixing.

(c) The container for volume change determination should be watertight, of constant cross-section and should maintain its shape during the period of testing. It should be not less than 12″ high and between 2″ and 3″ in diameter.

(d) The volume change determination should be as follows:

$$\text{Volume Change } (\%) = \frac{h_s - h}{h} \times 100$$

where:

h = grout column height at the time of casting

h_s = grout column height after initial set

Readers' Comment

DISCUSSION

by W. G. Corley*

The Apparent Modulus of Elasticity of Prestressed Concrete Beams Under Different Stress Levels

by

W. N. Lofroos and A. M. Ozell
(PCI Journal September 1959)

The authors have compared the results of their tests covering a 90-day period with the recommendations of the ACI-ASCE Joint Committee 323.[1] These recommendations are to be used to design structures for a useful life of a great many years. Troxell, Raphael, and Davis point out that one may expect only about 55 percent of the 20-year creep strain and about 56 percent of the 20-year shrinkage strain at the end of 90 days.[2] They go on to say that the creep strain at 90 days may range between 40 and 70 percent of the 20-year creep and the 90-day shrinkage strain may range between 40 and 80 percent of the 20-year shrinkage.

Since only slightly over half of the 20-year creep and shrinkage may be expected in 90 days, it appears unjustified to compare these test results directly with the Joint Committee recommendations. Furthermore, because of the possibility of large variations in the ratios of 90-day creep to total creep and of 90-day

*Graduate Student in Civil Engineering at the University of Illinois, Urbana, Illinois

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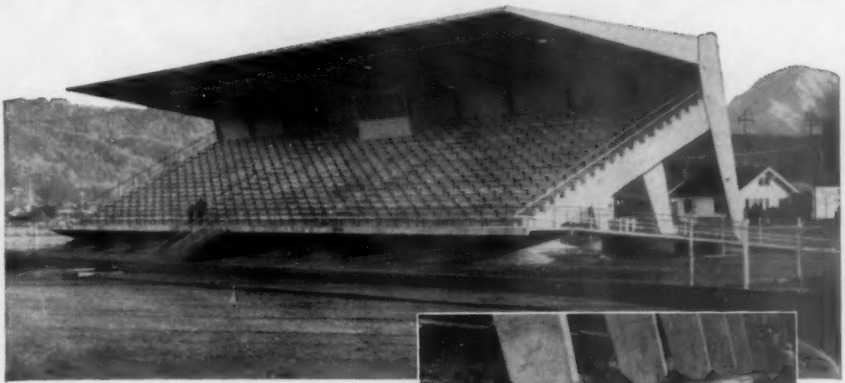

(Right) With soft underlying mud limiting foundation soil pressure to 500 psf, shell concrete footings were used over gravel to distribute load.

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The structure was assembled from precast, prestressed concrete units, drawn together by high-tensile-strength cable. Its slender appearance belies its ability to withstand earthquakes, 87-mph winds and as much as 95 tons of snow.

The structural frame was composed of eleven "F"-shaped units, each consisting of a slanted "backbone" column, a saw-toothed diagonal seat-supporting beam, and a cantilevered roof beam. All of these members were precast and prestressed—pre-tensioned on casting beds, and post-tensioned in assembly. Erected by crane, they were joined together with steel cables which were tensioned by hydraulic jacks to forces as high as 200 tons.

Seats were formed of prestressed L-shaped units placed on the saw-toothed members, and the roof consists of thin prestressed slabs resting on cantilevered beams.

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shrinkage to total shrinkage it appears risky to attempt to confirm or refute the Joint Committee recommendations on the basis of extrapolations from 90-day tests.

In their conclusion 3, the authors correctly point out that there appears to be no definite relationship between the "Apparent Modulus of Elasticity" and the cylinder properties. This points out the disadvantage of using an "Apparent Modulus" to determine the time-dependent deformations of a prestressed concrete beam. The "Apparent Modulus" is as much a function of loading and prestressing as it is of time and of the material properties of the beam. The use of an "Apparent Modulus" for the computation of time-dependent deformations in a prestressed concrete beam ignores the stress history of the concrete in the beam. Since the change in concrete stress may be rather large over a long period of time, it is important that the stress history be considered.

A preferable approach to the analysis of time-deformations in prestressed concrete beams is either the "Rate of Creep" method or the "Superposition" method. The "Superposition" method was first described by McHenry in 1943.[3] The development of the "Rate of Creep" method cannot be attributed to any one person but is well described in an article by Ross in 1958.[4] These two methods of analysis take into account the stress history of the concrete in the beam. Basically, each method relates the time-deformation relationship of concrete under a constant stress to that of concrete under a variable stress. If the time-deformation properties of concrete under a constant stress can be related to the cylinder properties, the time-deformation of a prestressed beam can be predicted.

There are a large number of variables which affect creep and shrinkage. At present, there is no entirely satisfactory way of taking the many variables into account. It is therefore necessary either to determine the time-deformation properties for the particular concrete to be used in the beam by special tests or else to make a reasonable estimate of the range of these properties.

The beams tested by the authors have been analyzed by the writer using the "Rate of Creep" method. The results of this analysis are presented below.

For simplicity, it has been assumed that creep, shrinkage, and relaxation all have the same relationship to time. The shape of this relationship has been taken as that found in the tests of reference (2):

$$\Delta = \log_e (t+1) \qquad (a)$$

where: Δ = fraction of the total creep, shrinkage, or relaxation
K = slope of the curve on a semilogarithmic plot
t = time in days

The magnitude of the shrinkage strain at the end of two years has been taken to be 300×10^{-6} and the magnitude of the relaxation was taken as 4% of the initial steel stress. The creep strain at the end of two years was estimated to be twice the instantaneous strain. The instantaneous modulus of elasticity was taken as that recommended by the Joint Committee:[1]

$$E = 1{,}800{,}000 + 500\, f'_c \qquad (b)$$

If it is assumed that the above amounts of shrinkage, relaxation, and creep take place in 2 years, equation (a) becomes:

$$\Delta = 0.152 \log_e (t+1) \qquad (c)$$

Using the above assumptions, the time-dependent deformations were computed and plotted with the test results. The results for beams 1 and 2 are shown in Figure 1 and the

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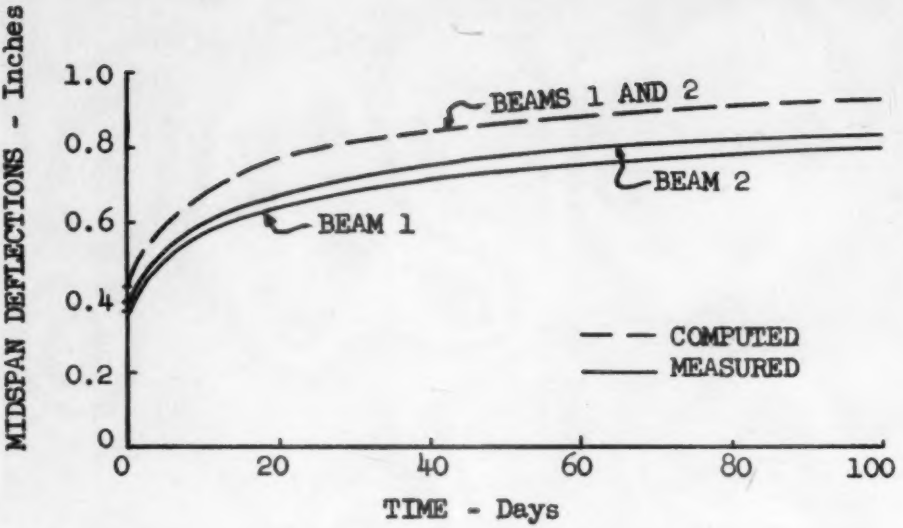

FIG. 1. COMPARISON OF MEASURED AND COMPUTED DEFLECTIONS FOR BEAMS 1 AND 2.

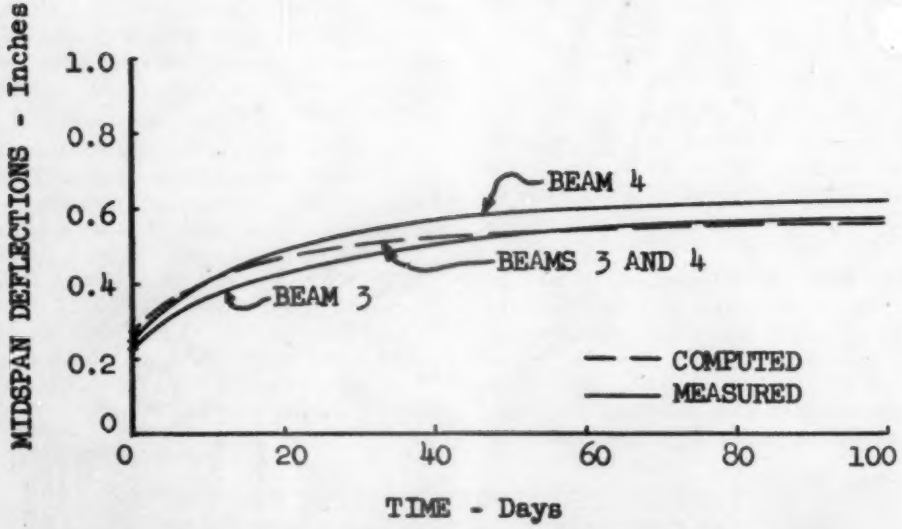

FIG. 2. COMPARISON OF MEASURED AND COMPUTED DEFLECTIONS FOR BEAMS 3 AND 4.

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results for beams 3 and 4 are shown in Figure 2. It can be seen from these figures that the rate-of-creep method gives consistent results with only one set of assumptions for the material properties. This results from the fact that it takes into account the stress history of the concrete as well as the cross sectional properties of the member.

1. ACI-ASCE Joint Committee 323, "Tentative Recommendations for Prestressed Concrete," Journal ACI, Proceedings V. 54, Jan. 1958.
2. Troxell, G. E., J. M. Raphael, and R. E. Davis, "Long Time Creep and Shrinkage Tests of Plain and Reinforced Concrete," Proceedings ASTM, V. 58, 1958, pp. 1101-1120.
3. McHenry, D., "A New Aspect of Creep in Concrete and its Application to Design," Proceedings ASTM, V. 43, 1943, p. 1069.
4. Ross, A. D., "Creep of Concrete Under Variable Stress," Journal ACI, Proceedings V. 54, March 1958, pp. 739-758.

Closure by Author

A. M. Ozell*

Mr. Corley's discussion and comments on the paper are interesting. Particularly his comparisons of the empirical values with the test results as presented in his Figures 1 and 2 are noteworthy.

In the original paper the intention was not to make a comparison of the 90-day test results with the values given in the ACI-ASCE Joint Committee 323 recommendations except to point out that the value of E as recommended by the Joint Committee seems to be high.

It is agreed that concrete shrinkage and creep undoubtedly continue significantly for long periods of time. However, the apparent modulus is primarily a function of creep only, since the apparent or sustained modulus for a constant sustained stress is defined as

$$E_{sus} = \frac{E_c}{1 + C}$$

where C is the ratio of creep strain to elastic strain. But in the case of the prestressed beam, the sustained prestress force is not constant since concrete creep tends to reduce camber growth by contributing to the loss of prestress. In addition, concrete shrinkage increases the loss of prestress.

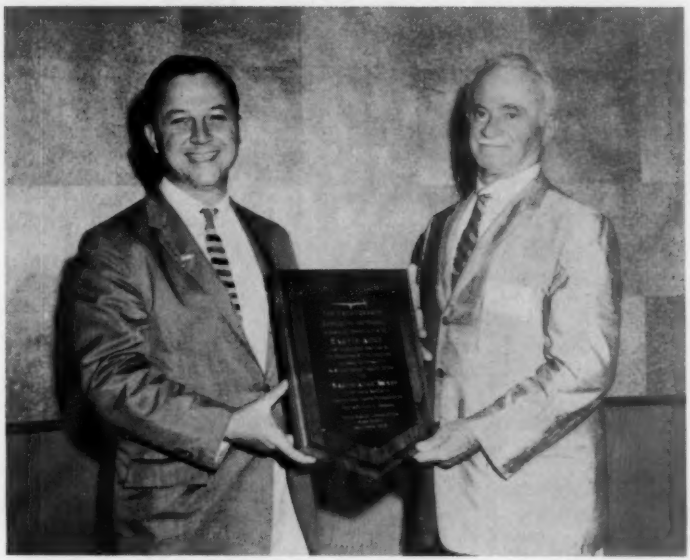
Tests show that camber growth reaches a maximum value relatively early (say about 100 days) and that the apparent modulus of elasticity for prestressed concrete beams will also reach a limiting value at that time.

With regard to Mr. Corley's statements referring to the need for considering the stress history, the method of using measured camber versus time relationship for determining the apparent modulus of elasticity for prestressed concrete beams automatically takes this account.

More comprehensive tests conducted at the University of Florida recently, on beams of similar dimensions as in the original series, have brought out additional information which will soon be published. It is hoped that through such efforts a better understanding of the behavior prestressed concrete members will be accomplished.

*Professor of Civil Engineering, University of Florida, Gainesville, Fla.

The Martin P. Korn Award



Past President Peter J. Verna presented a bronze plaque to Col. Martin P. Korn at the Fifth Annual PCI Convention. This plaque symbolized the establishment of the annual Martin P. Korn Award.

The Martin P. Korn Award of the Prestressed Concrete Institute was established at its Fifth Annual Convention at Miami Beach in honor of Martin P. Korn, first Executive Secretary of the Institute. This Award will be presented annually to the author of the best paper published in the PCI JOURNAL.

COMPETITION RULES

I Competition for the Martin P. Korn Award is restricted to members of the Institute.

II The Award consists of $100.00 in cash, with an appropriate engraved certificate. In case of more than one author, the cash will be divided, but each will receive a certificate.

III All original papers presented to the Institute by members in any grade, and published in the JOURNAL during the year for which the Award is given, are open to the Award provided that such papers have not been previously contributed in whole or in part to any other associations, nor have appeared in print prior to their publication by the Institute. Papers written jointly by members and non-members are not eligible. Papers to be considered for this year's Award are those that appear in the December 1959, March, June and September 1960 issues.

IV The Award, with certificate, is awarded to the author, or authors of a paper which shall be judged worthy of special commendation for its merit as a contribution to the advancement of prestressed concrete. The ceremony conferring the Award will be held at the 1960 PCI Convention in New York City Sept 27-30.

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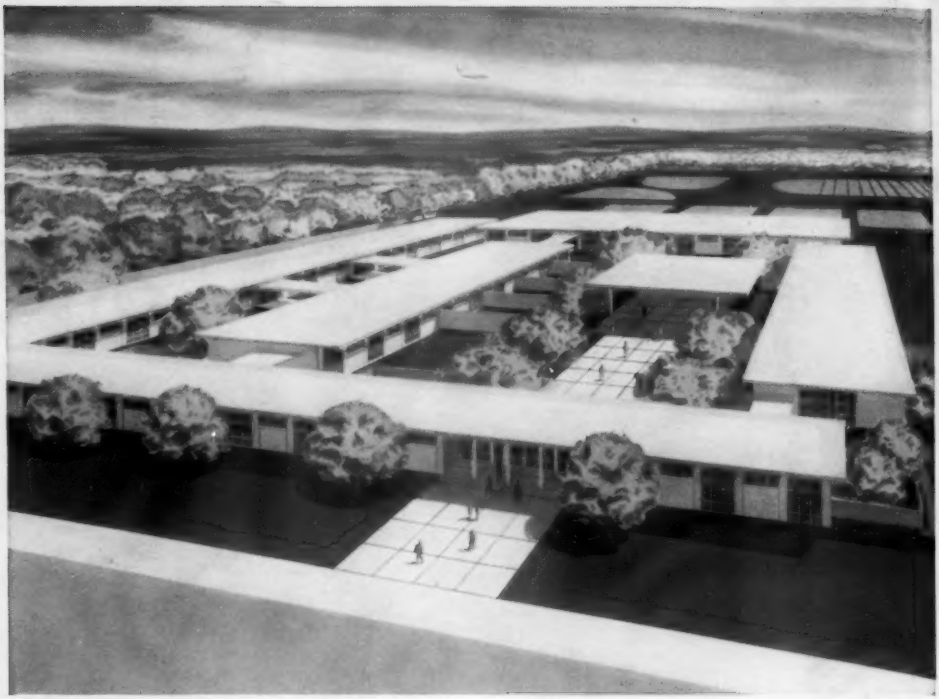

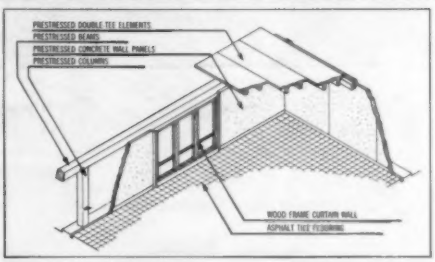

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